THE REGIONAL MUNICIPALITY OF HALTON

DESIGN UPDATE REPORT FOR THE OAKVILLE WATER PURIFICATION PLANT

MAY 1982

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Mr. R. W. J. Moore, P. Eng.
Director of Public Works

Regional Municipality of Halton
2316 South Service Road West
P.O. Box 7000
Oakville, Ontario
L6J 6E1

Dear Mr. Moore:

Re: Update Report on the Oakville Water Purification Plant

We are pleased to submit our update report on the Oakville Water Purification Plant.

During the preparation of this report, we have received excellent co-operation and assistance from members of the Region Public Works and Planning Departments, and we take this opportunity to thank these persons.

The report is presented in two forms, commencing with a detailed summary, and followed by the complete technical report including plans and figures. It is anticipated that this complete document will be useful as a permanent record for the continued use of the Region Engineering Staff. The report summary is presented separately for easy reference.

We trust this report will provide the Region with the necessary information for expansion of the Oakville Water Purification Plant. We would be pleased to discuss this report in detail with you or your staff, or provide additional information which might be required.

Yours very truly,

SIMCOE ENGINEERING LIMITED

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Report Summary

THE REGIONAL MUNICIPALITY OF HALTON

PR 669

DESIGN UPDATE REPORT
FOR THE
OAKVILLE WATER PURIFICATION PLANT

REPORT SUMMARY

1. PURPOSE

The purpose of this report is to summarize the present status of the Oakville Water Purification Plant and determine which components require improvement or expansion to meet the demands, both in terms of capacity and quality of finished water, for the ultimate population projected by current planning for the areas serviced by the plant.

2. SCOPE OF THE WORK

The scope of the work was determined by comprehensive Terms of Reference prepared by the Regional Municipality of Halton for the study. These Terms of Reference are appended to the main report.

3. CONSTRUCTION REQUIRED

On the basis of our review of the present status of the treatment plant facilities, and the future needs discussed in this report, additional works are needed at the Oakville plant to provide services and treatment process units not presently available, and to increase the production capacity of the plant.

Table S-1 summarizes these requirements for each of the plant components, and indicates the new construction timing required to meet the projected increase in demand.

TABLE S-1: REQUIRED CONSTRUCTION DATES BASED ON PROJECTED SYSTEM DEMANDS

ITEM	COMMENTS
INTAKES	Capacity sufficient for projected maximum demand. No improvement necessary.
LOW LIFT PUMPS	Capacity 109 000 m^3/d adequate until 1990.
FLOCCULATION	Capacity 109 000 m^3/d adequate until 1990.
SEDIMENTATION	Required immediately. Tube settlers required by 1993.
FILTRATION	Capacity 117 000 m^3/d adequate until 1993.
HIGH LIFT PUMPS	Capacity 82 000 m^3/d adequate to 1984. Pump replacement adequate to 1991. New station required in 1991.
CHEMICAL BUILDING	Required immediately.
CONTROL SYSTEM	Required immediately.
POWER SYSTEM	Replacement prior to 1986. Additional capacity required in 1993.
ADMINISTRATION BUILDING	Required immediately.
WASTEWATER TREATMENT	Required immediately.

4. WORK RECOMMENDED TO UPGRADE OPERATIONS

These improvements will provide a more secure system, in terms of standby capability; improve the efficiency of operation of the plant overall; provide the necessary upgrading of the process to handle present and projected future treatment requirements; and ensure the plant can properly provide water to present and projected development.

Phase 1 of the recommended program of construction includes:

- a new sedimentation tank to provide adequate treatment at times of high raw water turbidity;
- improvements to meet the projected maximum requirements, and consolidation of the chemical systems into a new chemical equipment area;
- interior reconstruction of the present administration building to provide essential personnel facilities, a new laboratory area, and a new control room;
- improvements to the existing high lift pumping station;
- construction of a new transformer substation to replace the existing one being removed from service by Oakville Hydro;
- a wastewater treatment system;
- improvements to and duplication of the raw water discharge header;
- a computer-based supervisory and control system.

The recommended construction schedule for these works is 1983/84, and the estimated total cost for this stage of the construction (Phase 1) is $5 300 000.

5. WORK RECOMMENDED TO INCREASE PLANT CAPACITY

The improvements discussed above will be adequate to meet the projected maximum needs of the Oakville plant in terms of auxiliary, or non-production facilities. The capacity after these improvements are completed will be 109 000 m^3/d, and it will be necessary, therefore, to provide additional process units to bring the net production capacity up to the projected maximum requirement of 161 600 m^3/d.

The next stages of the improvements required should be constructed over the period 1988-92 to meet the projected demands, such construction to take place in two stages - Phases 2 and 3 of the overall program.

Phase 2 includes:

- new flocculation tanks to meet the projected maximum production requirements;
- an additional raw water pump in the low lift station, and a new raw water screen, both to meet the projected maximum production requirements;
- a second high lift pumping station, including a surge suppression tank to protect the distribution system, and two pumps to meet the projected maximum production requirements.

The recommended construction schedule for these works is 1988/89, and the estimated total cost for this stage of the construction (Phase 2) is $3 050 000.

TABLE S-2: ESTIMATED COSTS

PHASE 1	
Sedimentation tank	1 450 000
Chemical building	500 000
Administration building	750 000
New high lift pumps and improvements	400 000
New substation	400 000
Wastewater system	850 000
Control system	750 000
Low lift system improvements	200 000
PHASE 1 - TOTAL	$5 300 000
PHASE 2	
Flocculation tanks	2 100 000
Low lift pump and screen	150 000
New high lift station	800 000
PHASE 2 - TOTAL	$3 050 000
PHASE 3	
New filters	1 800 000
Tube settlers	850 000
2000 kVA transformer	100 000
PHASE 3 - TOTAL	$ 2 750 000
TOTAL - all phases	$11 100 000

Phase 3 includes :

- new filters to meet the projected maximum production requirements;
- installation of tube settlers in the sedimentation tank to uprate this unit to meet the projected maximum production requirements;
- addition of a 2000 kVA transformer to provide security and to meet the projected maximum production requirements.

The recommended construction schedule for these works is 1991/92, and the estimated total cost for this stage of the construction (Phase 3) is $2 750 000.

Table S-2 opposite shows the estimated costs for the various components of each phase of the expansions.

6. ENVIRONMENTAL ASSESSMENT

Under regulations made in 1980, the Environmental Assessment Act has extended to undertakings initiated by municipalities.

We believe that virtually all of the new works or additions to existing facilities will require approval under the Act, or exemption under Regulations. Phase 2 and Phase 3 works will probably be covered by Class Assessments, although it is not clear at present what classes will be established as the Ministry of the Environment have yet to publish the final documentation for these procedures.

6.1 Ontario Regulation 293

Revised Regulation of Ontario 1980, known as the Municipal Regulation, contains a number of important provisions particularly with regard to exemptions from the Act. Section 5.5(a) deals with undertakings where certain specified commitments have been made prior to June 3, 1980, and where the municipality has committed a minimum of 25% of the estimated cost of the work by Decmber 31, 1983.

Section 5.5(c) deals with undertakings which comprise expansions of an "existing water works or water treatment facility" where "the expansion is located at the site of, or adjacent to an existing water treatment facility".

Either or both of these exemption provisions could apply to the works proposed under the first phase of the construction program referred to in Section 14 of our report. It will be necessary for review to be made of the relevant past events and commitments to determine if either or both of these exemption provisions apply.

6.2 Future Works

For the second and third phases of the expansions programme proposed, namely for the period 1988 - 1992, the exemption provisions referred to above will have expired. By that time the Municipal Class Assessment will be in place which will determine how the Region will be required to respond to the Act. Based on the most recent draft of the Class Assessment document available to us, we would expect the works to be carried out at this stage to be subject to a "screening" process. The screening criteria are proposed in the Class E.A. document, and for more information on these criteria reference should be made to that document.

6.3 *Recommendation*

Because the provisions of the Environmental Assessment Act can have very significant effects on the development of the expansions of the Oakville Water Treatment Plant, we recommend that for the next stage of the expansions programme the Region carefully review the Environmental Assessment Act, including the most up-todate Regulations and establish whether or not the Region is in a position to qualify for an exemption under the "Grandfather" provisions. If those provisions can be satisfied, and if the Region as a matter of policy wishes to do so, a submission should be made to the Environmental Approvals Branch of the Ministry of the Environment for confirmation that the works would be exempt.

6.4 Summary

A number of important consequences can be expected to follow the implementation of the Environmental Assessment Act with regard to the future development of the Oakville water purification plant. The preparation of environmental assessments will require time, and the reports needed at the pre-design stage can be expected to address more areas (e.g. alternatives) than has been necessary hitherto. Further, the approvals processes themselves will probably take more time and in some instances a Public Hearing might be required.

The estimates of cost which we have presented are based on past experience and do not allow for any additional construction or engineering and legal costs which might result from satisfying the Environmental Assessment Act.

7. ACKNOWLEDGEMENTS

This report was prepared by Simcoe Engineering Limited, with the assistance of a number of people outside the company, with regard to data collection and miscellaneous items. We wish to acknowledge the direct and indirect assistance of the staff of the water plant who, among others, supplied important information.

We wish to express our appreciation to Mr. R. W. J. Moore, Mr. C. F. Kong, Mr. E. B. Robertshaw, and other Regional staff who have assisted in the development of the background information. A special word of thanks is given to Mr. R. M. Schwandt of Oakville Hydro for his assistance with respect to information and projected planning for the hydro system.

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SECTION 1

INTRODUCTION

1.1 PURPOSE OF THE REPORT

In the light of the most recently completed planning and servicing studies, and servicing commitments made to developers by the Regional Municipality of Halton, it became necessary to revise the presently proposed expansion plans for the Oakville Water Purification Plant.

The purpose of this report, therefore, is to summarize the present status of the water purification plant, and determine which components require improvement or expansion to meet the projected maximum demands, both in terms of capacity and quality of finished water delivered to the distribution system.

1.2 SCOPE OF THE WORK

Comprehensive terms of reference were prepared by the Region (Appendix 1) for the scope of the work to be included. The work described required a full and complete report on future development of the treatment plant, recommended staging of the various improvements or extensions to the plant, and estimated costs for the construction programme recommended.

1.3 SOURCES OF INFORMATION

Primary sources of information referenced for this study were:

1. Report on the Operation, Maintenance, Instrumentation and

Capacity of the Oakville Water Treatment Plant - April 1976,

Prepared by James F. MacLaren Limited

2. Pre-design Report on the Expansion of the Oakville Water Treatment Plant - March 1977

   Prepared by James F. MacLaren Limited

3. Report on Waterhammer Protection for the High Lift Pumping System of the Oakville Water Treatment Plant - November 1978

   Prepared by James F. MacLaren Limited

4. Report on the Extension of the 1828 mm Diameter Intake at the Oakville Water Treatment Plant - December 1978

   Prepared by James F. MacLaren Limited

5. Report on the Modernization of Supervisory Control for the East District Water Supply and Distribution System - November 1980

   Prepared by James F. MacLaren Limited

6. Master Servicing and Financing Scheme for the Town of Oakville (Draft) - February 1981

   Prepared by Currie, Coopers & Lybrand Limited

7. Water Distribution Study - Town of Oakville - September 1981

   Prepared by Simcoe Engineering Limited

Information was provided by Region of Halton staff at meetings held during the course of the work, and additional information was obtained from plant operating records and inspections of the plant.

SECTION 2

WATER PRODUCTION REQUIREMENTS

2.1 FINISHED WATER

The Water Distribution System Study for the Town of Oakville, prepared in 1981, and accepted by Regional Council on November 4th, 1981, comprehensively addresses the future quantities of water required. Reference should be made to that report for detailed analyses and the basis of calculation used to determine the future finished water requirements.

The essential conclusion of that report with regard to plant capacity was that, based on the current development planning, the future water requirements would be as shown in Table 2-1:

TABLE 2-1: PROJECTED FUTURE TREATED WATER DEMANDS

YEAR	AVERAGE m^3/d	MAXIMUM m^3/d
1986	48 105	86 476
1991	57 495	103 368
1996	68 219	122 658
2001	77 106	138 629
PROJECTED MAXIMUM	89 899	161 616

2.2 RAW WATER

The quantity of raw water requiring treatment is always greater than the quantity necessary to satisfy the demands of consumers connected to the distribution system, since the treatment process itself consumes some treated water.

Typically, these uses include water for sanitary use, house-cleaning, dilution water for certain chemical systems, and for backwashing filters (the largest in-plant use).

Experience at other Lake Ontario treatment plants shows these extra water uses are approximately five per cent of finished water quantity. Therefore the raw water pumping and treatment requirements are five per cent higher than shown above:

TABLE 2-2: PROJECTED FUTURE WATER TREATMENT REQUIREMENTS

YEAR	AVERAGE m^3/d	MAXIMUM m^3/d
1986	50 500	90 800
1991	60 400	108 500
1996	71 500	128 800
2001	81 000	145 600
PROJECTED MAXIMUM	94 400	169 700

Figure 1 following summarizes the projected changes in demand for the future, and indicates the capacity of various units of the present plant.

2.3 WATER PRODUCTION - SUMMARY RECOMMENDATIONS

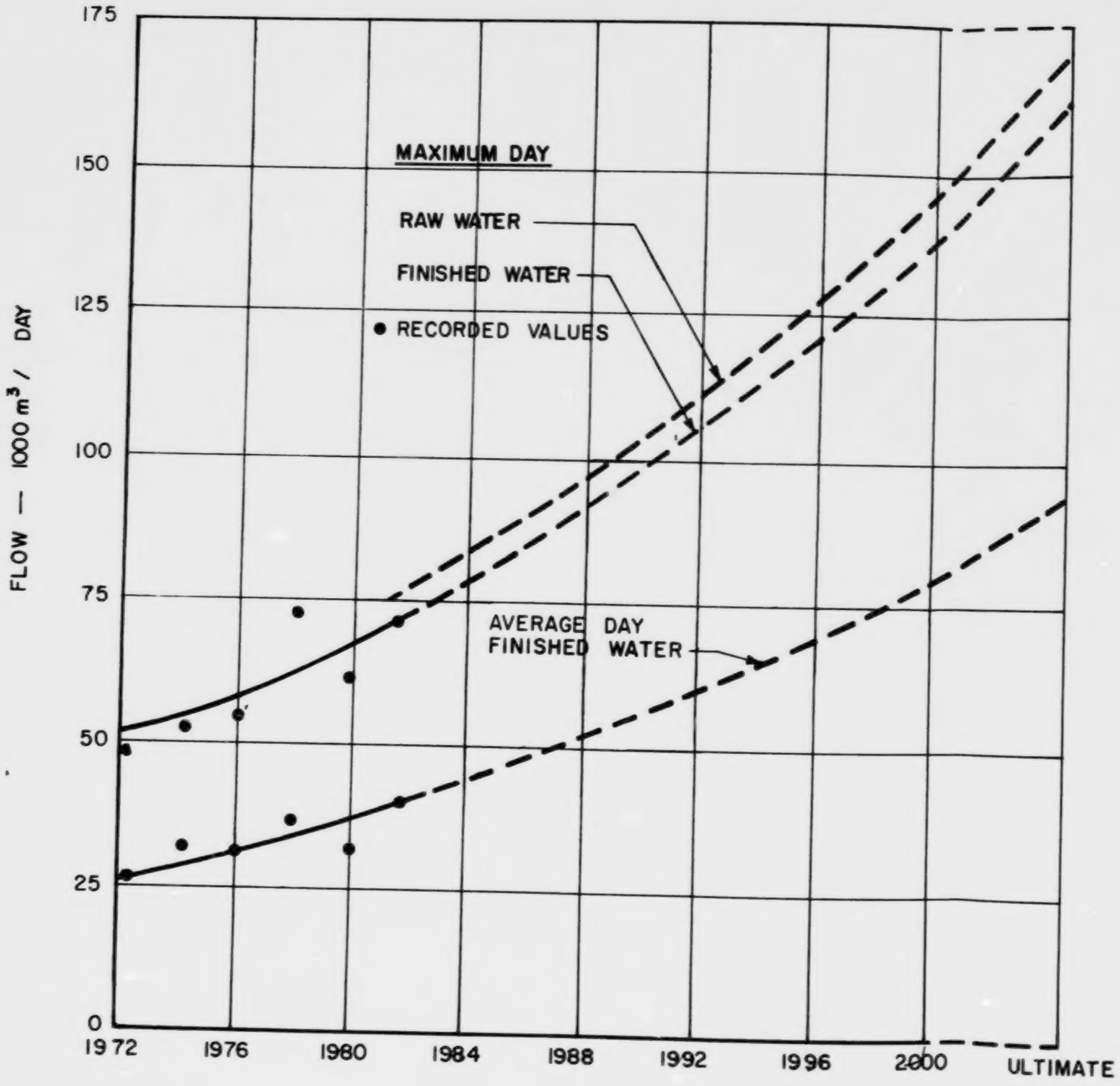
As described in our study on the water distribution system, we recommend that operation of the treatment plant (and remote pumping stations) be at an approximately uniform rate at all times, allowing the in-system reservoirs to meet peak flow requirements. This system of operation is the most commonly employed in major systems, having the advantage that the treatment plant need only *supply the maximum day requirement (frequently 40-50% less than maximum hour or peak flows).*

We further recommend that the treatment capacity of the Oakville Water Purification Plant be not less than 169 700 m^3/d*.*

REGIONAL MUNICIPALITY OF HALTON

OAKVILLE WATER PURIFICATION PLANT: CAPACITY SUMMARY

PRODUCTION PROJECTION



CAPACITY DATA

INTAKES			
762 mm dia	appr.	60 000 m^3/d	TOTAL CAPACITY
1828 mm dia	appr.	300 000 m^3/d	>300 000 m^3/d
SCREEN			TOTAL CAPACITY
No. 1	--	109 000 m^3/d	109 000 m^3/d
LOW LIFT PUMPS			
No. 1	--	54 500 m^3/d	FIRM CAPACITY
No. 2	--	27 250 m^3/d	109 000 m^3/d
No. 3	--	54 500 m^3/d	
No. 4	1976	27 250 m^3/d	
	INSTALLED	163 500 m^3/d	
FLOCCULATION			TOTAL CAPACITY
No's. 1 & 2	1979	109 000 m^3/d	109 000 m^3/d
FILTERS			TOTAL CAPACITY
No's. 1-4	1981	53 300 m^3/d	117 000 m^3/d
No's. 5-8	1981	63 700 m^3/d	
HIGH LIFT PUMPS*			
No. 1	1977	34 000 m^3/d	
No. 2	1953	7 000 m^3/d	FIRM CAPACITY
No. 3	1961	22 500 m^3/d	82 000 m^3/d
No. 4	1971	29 500 m^3/d	AT SYSTEM HEAD
	INSTALLED	93 000 m^3/d	

*High lift pump individual ratings at 70 m head: firm capacity at system head. Refer to report text.

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FIGURE 1

SECTION 3

WATER QUALITY ASSESSMENT

3.1 GENERAL

The quality of the raw water drawn from the Oakville intake is typical of that at most other Lake Ontario locations with the exception of turbidity.

Historical data indicate that the quality of the water in all respects complies with established guidelines for raw water supplies for municipal treatment and distribution, and no unusual corrective treatments are required.

The data show that the raw water does not meet current standards for finished water bacteriological quality or turbidity, but is satisfactory in all other aspects.

The raw water presently supplied to the plant cannot be compared in great statistical detail with that which occurred prior to the intake extension, as insufficient time has elapsed since the extension for a reliable data base to be established for the water quality from that extension.

The intake now draws water from a greater depth in the lake, and a greater distance from shore than before. In this respect water quality should be more constant and less subject to changing weather conditions affecting the water. In addition, contamination from local land based sources should be less frequent.

The basic character of the water will probably be almost

TABLE 3-1

1977-81 DATA: TURBIDITY RANGE OBSERVED - RAW WATER

		JAN.	FEB.	MARCH	APRIL	MAY	JUNE
1977	RANGE	2-8	2-10	2-35	4-20	3-7	2-6
	HIGHS	-	27/35	49/360	100/200	-	-
1978	RANGE	4-12	2-9	1-10	10-100	3-16	2-8
	HIGHS	-	-	125/170	-	30/35	-
1979	RANGE	1-10	1-3	1-20	1-50	1-10	1-3
	HIGHS	35/150	-	-	220/500+	-	-
1980	RANGE	1-14	1-7	1-5	1-14	1-5	1-3
	HIGHS	52	11	-	20	17	-
1981	RANGE	1-7	1-14	1-5	1-10	1-5	1-6
	HIGHS	-	18/19	20	-	-	-

		JULY	AUG.	SEPT.	OCT.	NOV.	DEC.
1977	RANGE	2-10	2-8	2-20	1-5	2-10	4-50
	HIGHS	-	12	25/44	10	18	-
1978	RANGE	2-5	3-10	3-15	1-4	1-4	1-3
	HIGHS	-	-	25	-	11	-
1979	RANGE	1-3	1-3	1-4	1-3	1-10	1-10
	HIGHS	9	-	-	7	-	90/120
1980	RANGE	1-3	1-10	1-3	1-7	1-4	1-4
	HIGHS	-	-	-	-	-	-
1981	RANGE	1-4	1-7	1-5	2-10	2-6	1-5
	HIGHS	11/44	-	-	-	18/14	15

NOTE: 'HIGHS' indicates the (two) highest recorded values in the month. Where no extreme values occur, no highs are indicated.

unchanged as it relates to the problems, or lack thereof, in its treatment. In general, the raw water will be suitable for conventional treatment, i.e. suspended solids removal and disinfection.

3.2 TURBIDITY

Water quality at Oakville has one major difference from that experienced at most other Lake Ontario sources. Sixteen Mile Creek discharges adjacent to the treatment plant which under certain conditions (discussed in more detail in Section 4) exerts an extreme effect on the raw water turbidity in the general vicinity of the intake.

Table 3-1 opposite gives a summary of raw water turbidity data abstracted from plant records for the period 1977-1981. For many months the raw water turbidity varies over a fairly small range with occasional highs. Late Winter and early Spring months are notable exceptions with extremely high turbidities observed. Data prior to 1977 also exhibited similar Spring peaks.

At present, a raw water sampling and turbidity monitoring system is in operation drawing raw water from a location approximately 360 metres further into the lake beyond the end of the existing 1828 mm intake.

Preliminary data from this sample location have not indicated any noticable improvement in raw water turbidity when compared with historical data.

3.3 Special Characteristics

In 1981, the MOE completed a study of Metropolitan Toronto and Area water treatment plants to analyse raw and treated water for organochlorine pesticides and dioxin. The results obtained for the Oakville plant found all materials to be below the present detection limits of 1-2 nanogram/litre.

In April, 1980, tests were conducted to determine the Tritium concentration in the raw water. The analysis indicated a tritium concentration equivalent to 647 picaCurie/L or 0.01 per cent of the MPC_w for tritium. MPC_w is the maximum concentration of radionuclides in water which, if continuously maintained, would result in a person who consumes 2.2 litres/day receiving an annual whole body radiation dose of 500 millirem - the safe level as determined by the Atomic Energy Control Board.

The raw and treated water from the Oakville plant was analysed in 1979 for asbestos fibres, and both samples were found to be below the detection limit of 0.2 million and 0.4 million fibres/litre for treated water and raw water, respectively.

During 1977, 1978 and 1980, samples of raw and treated water from the Oakville plant were analysed regularly for total trihalomethane concentration. Average treated water values were 25 microgram/litre and a maximum of 40 microgram/litre was found. The raw water contained no trihalomethanes. The current guideline maximum concentration is 350 microgram/litre.

SECTION 4

WATER TREATMENT PROCESS OPTIONS

4.1 GENERAL

Based on a comparison between raw water quality and the finished water quality guidelines established by the Ontario Ministry of the Environment, and the Federal Department of Health and Welfare, it is apparent that the raw water quality may be characterized as being of high quality.

The necessary treatment to produce finished water meeting these guidelines would be disinfection and reduction of turbidity to the objective value. However, it is highly likely that guidelines for water quality will include limitations on materials presently not covered by existing guidelines, i.e. total halomethane concentration, and the treatment processes adopted as part of a major plant expansion should recognize this probability.

4.2 TURBIDITY REDUCTION

It is apparent from the water quality data that one of the primary requirements of the overall treatment process - from lake to consumer - is a reduction in turbidity. This reduction can be effected in two ways - by sedimentation followed by filtration, or by filtration alone.

For filtration alone, it is necessary for the raw water turbidity to be sufficiently low that the design of the filters is economical. The most commonly accepted criteria for filtration alone to be economical are that filter runs of normally at least twentyfour hours can be achieved, with effluent turbidity meeting the normal finished water criterion of 1.0 NTU, and at a filtration rate greater than about 9 m/hr. If lesser filtration rates are required to achieve these results then the filter area needed increases which becomes more expensive. If shorter filter runs occur, then greater quantities of washwater are required, and the net daily output of the filters is reduced. Typically, a raw water turbidity of less than 15 NTU is required for filtration alone to be the most economical treatment. During the period 1977-1981, values greater than 15 NTU were recorded 6.7 per cent of the time: values greater than 50 NTU were recorded 1.7 per cent of the time.

There are two alternative approaches to providing adequate treatment: either settling tanks can be added to the treatment process to reduce the turbidity of water reaching the filters; or the raw water provided to the treatment process can be taken from a point where such high turbidities do not occur.

In general, the turbidity of water in Lake Ontario is less than 5 NTU. However, the water quality in the vicinity of the Oakville treatment plant is affected by discharges from the adjacent Sixteen Mile Creek which produce an extremely high turbidity plume entering the lake a short distance east from the Oakville plant, under certain conditions.

The direction of travel, and degree of dispersion of this plume is affected by a large number of factors, including lake bed topography, lake currents, prevailing winds, variations in temperature between different parts or depths of water in the lake and the temperature of the plume leaving Sixteen Mile Creek. Since very few data are available for these parameters, it is virtually impossible to predict with any certainty where the plume will travel in the Lake, or what the quality of the water will be at any particular location.

A review of all of the available aerial photographs obtained from Northway-Gestalt Inc. of the Oakville plant area - seven in all - showed clear indications of a turbid plume in three instances. The plume was evident as travelling west, east, and south in these three cases.

In terms of comparative cost for the two alternatives, we estimate the initial cost for a sedimentation tank to be $1.45 million for the tank alone, plus an additional $0.85 million for the installation of inclined tube settlers which effectively raise the capacity of the proposed tank to the ultimate capacity (as discussed in Section 7).

The most recent intake extension serves as a guide to the probable cost of a further extension. Making allowance for higher costs for work in deeper water and further offshore than that extension, and updating the 1981 reported costs for the intake extension, we estimate that an extension of approximately 500 metres could be constructed for the same cost as the sedimentation tank discussed above.

We believe that, on the basis of the physical evidence, it is not possible to identify a location within the lake at a reasonable distance from the present intake where the raw water turbidity can be guaranteed unconditionally to be below the operating limits for filtration alone. However, the performance of a sedimentation tank can be predicted in that it is governed by known and predictable physical laws, i.e. material suspended in the raw water will settle out when the water is in a quiescent state.

4.3 DISINFECTION

A major objective of the treatment process is to provide a bacteriologically safe drinking water. Many methods may be used for disinfection; the most common and practical at a large treatment plant are the use of chlorine, chlorine dioxide, or ozone, either alone or in some combination.

4.3.1 Chlorine

Chlorine is the disinfectant used for water treatment virtually worldwide. Consequently, a considerable amount of information is available of the advantages and disadvantages of chlorine. Its major advantages when compared with the other disinfectants are that there are very well developed operating practices for the safe handling of the material, and its cost is considerably lower than the alternatives. Its major disadvantage is that organic material present in the water being chlorinated is converted to tri-halomethanes (T.H.M.) which may have undesirable biological properties. The finished water from the Oakville plant has been analysed by the Ontario Ministry of the Environment, and the levels of tri-halomethanes were determined to be substantially below the recommended maximum levels.

4.3.2 Chlorine Dioxide

The major advantage of chlorine dioxide is that it is able to break down many organic compounds which if chlorinated would produce undesirable tastes or odours in the water, and that it is reported to be able to disinfect water containing organic material without producing tri-halomethanes.

4.3.3 Ozone

Ozone is the most powerful oxidant suitable for use in water treatment, and therefore the most effective in breaking down organic contaminants into smaller molecules, and achieving a very high degree of bacterial and viral destruction. A substantial disadvantage of ozonation is that it is a very energy intensive process with substantially higher costs both for operation and maintenance than for chlorination systems.

Since ozone is highly reactive, it is not possible to carry a disinfectant residual into the distribution system, as required by the Ministry of the Environment, by using ozone alone.

4.3.4 Discussion of Disinfectants

There is a great deal of published information on the health and health-related aspects of the various disinfectants. Chlorine has the disadvantage that it can generate the potentially hazardous THM's. Chlorine dioxide does not produce these materials, but evidence from recent studies suggests that the potential mutagenic/carcinogenic effect of organics oxidized by chlorine dioxide may be greater than when oxidized by chlorine. Chlorine dioxide itself has potential adverse effects, as illustrated by limits imposed by several European countries to permit a maximum dose of approximately 0.5 mg/L (which is almost certainly not sufficient for adequate disinfection of Lake Ontario raw water).

Ozonation has been demonstrated to be highly effective in breaking down organic compounds, but may possibly produce undesirable by-products such as the toxic epoxides. Studies have demonstrated in some instances a much higher THM concentration in water chlorinated after ozonation than water treated by chlorine alone.

We recommend the continued use of chlorine for disinfection, with chlorine dioxide maintained as a useful addition for taste/odour control.

4.4 ORGANICS CONTROL

The objectives of this aspect of the treatment process are to reduce to appropriate levels, or eliminate completely, various organic materials present in the water which are undesirable for any of the following reasons:

a) they are potentially biologically hazardous;
b) they contribute to unwanted tastes or odours in the finished water;
c) they can be converted to compounds in either of the above categories by reactions normally occurring in the treatment process.

At present, considerable research effort is being devoted to investigation of the best methods for organics control. Chlorine dioxide and ozone have been considered for chemical destruction of these materials as well as disinfection, and activated carbon adsorption and biological methods have been considered for removal of materials.

As discussed in the previous section, we consider the use of chlorine and chlorine dioxide to represent the safest method of disinfection. The available practical and proven methods for organics control, therefore, are reduced to one - the use of activated carbon.

There are two distinctly different processes involving the use of activated carbon - powdered activated carbon added at the pre-treatment stage, and granular activated carbon used either as part of the usual filtration process or as a separate post-filtration process.

Granular activated carbon will absorb most organic materials present in the finished water, but when used as a post-filtration stage, is expensive since additional contacting tanks or beds must be constructed, as well as the means of pumping the previously filtered water through the carbon beds. In addition, since activated carbon will destroy chlorine residuals, it is possible for biological growths to occur within the carbon bed and also possible for waste materials from these growths to add to the water under treatment toxic materials which would otherwise not be present.

Powdered activated carbon will operate in essentially the same manner by adsorbing organics. When used as a pre-treatment, by addition to the raw water, however, it has the substantial advantage that it can be used to remove organics prior to chlorination, thereby eliminating the possibility of THM formation. This approach is generally not practical where filtration without sedimentation is practised, since the carbon added is an extra solids load to the filters thus reducing their overall capacity. Where sedimentation is incorporated into the treatment, no such problem arises.

4.5 RECOMMENDED PROCESS

On the basis of the preceding discussion, the process required will depend on the raw water turbidity. During high turbidity periods greater quantities of aluminum sulphate and (possibly) polymer will need to be added to produce settleable material, and longer flocculation times are required. During low turbidity periods, coagulant doses and flocculation times can be reduced.

The aluminum sulphate and polyelectrolyte doses required for effective treatment can only be determined by evaluation of the process performance at any time. Typically, 'low turbidity' operation will require alum doses in the range 5-10 mg/L and polymer doses in the range 0.2-0.5 mg/L. 'High turbidity' operation will require alum doses in the range 30+ mg/L and polymer doses in the range 0.5-1.0 mg/L.

The doses of chlorine for pre- and post- chlorination should be determined on a regular basis consistent with the objectives of producing a measurable chlorine residual at entry to the filters (to prevent biological activity in the filters), and *producing the desired finished water residual - typically in the range 0.3-0.5 mg/L.*

Chlorine dioxide should be added only as required to control tastes and odours and the dose should be limited to below 0.5 mg/L. It may be added either at a pre-filtration or post-filtration stage in the treatment.

Fluoridation should be at the appropriate dose required for effective protection, typically 1.0-1.2 mg/L, adjusted seasonally.

Activated carbon, if required, should be added prior to sedimentation. Based on experiences with similar raw waters a dose of approximately 5 mg/L will control organics present at normal levels.

SECTION 5

INTAKES

5.1 PURPOSE

Intakes are required to transfer water from Lake Ontario into the intake well and to the low lift pumping station for delivery to the treatment modules. Capacity is developed by drawing down the level in the intake well to below the level of the lake, thus causing a positive hydraulic grade line.

5.2 EXISTING FACILITIES

Presently there are two existing intakes located approximately south of the plant:

1. 762 mm diameter concrete pipe installed in 1947-49, extending 760 metres into Lake Ontario, with a 3.7 metre square intake crib submerged 7.6 metres at a low lake level of 74.98 metres;
2. 1828 mm concrete pipe installed in 1969, extending 458 metres into Lake Ontario, with a 3.7 metre square intake crib submerged 4.1 metres. This intake has recently been extended 350 metres in a southeast direction to provide water from a deeper location, and one possibly less affected by the discharge plume from Sixteen Mile Creek.

A raw water sampling and turbidity monitoring system was in operation, drawing raw water from a location approximately 360 metres further into Lake Ontario, beyond the exising 1828 mm intake. The purpose of this system was to provide data pertinent to the decision on further extension of that intake to improve raw water quality.

5.3 LAKE LEVELS

The capacity of an intake facility is directly related to the lake level which can be considered for the evaluation of that facility. In general, the lake levels vary seasonally and also fluctuate in approximately seven-year cycles. These seasonal and cyclic fluctuations can result in severe restrictions on intake capacities.

The maximum and minimum lake levels experienced in the vicinity of the Burlington Water Purification Plant, which levels can be assumed to be almost identical for Oakville, are summarized in Table 5-1:

TABLE 5-1: SUMMARY OF EXTREME LAKE LEVELS

Condition	Time	Level* metres
Instantaneous High	June 8, 1952 - 1600 hours	75.96
Maximum Day**	June 3, 1952	75.93
Maximum Month***	June 1952	75.88
Instantaneous Low	December 26, 1934	73.62
Lowest Month***	December 1934	73.86

* All levels converted to geodetic datum
** Average value over one 24-hour period
*** Average value over one 30-day period

Prior to the completion of the St. Lawrence Seaway, levels in Lake Ontario were not controlled and the expected minimum level would correspond to the 1934 extreme. To a large extent, the Seaway system now controls these levels and this level is no longer totally a function of climatic conditions.

Information from Environment Canada, St. Lawrence Seaway Division, indicated that the future minimum lake level during summer should not be less than 74.26 metres geodetic. It has been indicated to us, however, that levels fell below this limit in 1964 and 1965 in the winter, and could occur again especially in the December to April period. The following values are deemed to represent a reasonable basis for evaluation purposes:

Minimum summer time level	-	74.26 metres
Minimum winter time level	-	73.62 metres
Maximum level	-	75.96 metres

Minimum summer and winter values are important since they are the basis of establishing the respective summer and winter capacities of the intake system.

5.4 INTAKE CAPACITY

On the basis of these levels, and assuming the maximum recommended drawdown in the intake wells of the low lift pumping station, for the type of pumps used, we calculate the capacity of the 1828 mm diameter intake to be approximately 300 000 m^3/d at the summer low level, and approximately 240 000 m^3 at the winter low level.

Since the projected maximum requirement for raw water is 169 700 m^3/d, it is apparent that sufficient intake capacity exists to meet all future demands.

5.5 INTAKES - SUMMARY RECOMMENDATIONS

We recommend that the larger diameter intake be used at all times, since its location in Lake Ontario is expected to result in higher quality raw water being provided to the treatment plant.

SECTION 6

LOW LIFT PUMPING STATION

6.1 PURPOSE

The screen and low lift pumps are located in a pumping station adjacent to the existing high lift pumping station. The screen is used to remove large debris from the raw water to protect the pumping and other equipment from mechanical damage. The low lift station is required to lift water via the raw water header to the treatment modules.

6.2 CAPACITY REQUIRED

The required output from this station must be sufficient to meet the maximum raw water demand of 169 700 m^3/d. In keeping with generally accepted good practice, this capacity should be available with the largest unit out of service.

6.3 EXISTING EQUIPMENT

The existing low lift pumping station is designed as a two part station with allowance for dual raw water screens. At present, there is one travelling screen and four low lift vertical turbine pumps, all installed on the west side of the station. The raw water discharge header is a 1050 mm diameter line which has a carrying capacity in excess of the maximum demand.

Table 6-1 shows the capacities of the existing pumps.

TABLE 6-1: EXISTING LOW LIFT PUMPING STATION

PUMP #	MAKE	CAPACITY
1	Worthington	54 500 m^3/d
2	Worthington	27 250 m^3/d
3	Worthington	54 500 m^3/d
4	Worthington	27 250 m^3/d

The total installed capacity is thus 163 500 m^3/d, and the firm capacity is 109 000 m^3/d. The existing equipment, therefore, is not adequate to meet the projected maximum demand.

We recommend the addition of a new 54 500 m^3/d pump on the east side of the station. The pump should be located as determined to be the optimum location by the 1979 University of Waterloo report prepared after a hydraulic model test of the Oakville intake well.

The stated capacity of the existing travelling water screen is 109 000 m^3/d, and since this is insufficient to meet the projected maximum demand, we recommend a second 109 000 m^3/d screen be installed in the low lift pumping station, in the existing screen well for the east side of the pumping station.

6.4 RAW WATER SYSTEM

The present low lift station is connected to the process units by a hydraulically adequate 1220 mm diameter discharge header. There is an existing 914 mm line connected from this header just south of the main entrance, through the administration building, and terminating just north of the existing filters. In keeping with the practice of providing duplicate or back-up systems for all essential plant components, a new header should be provided from the existing low lift station, joining that station to the existing 914 mm line. The connection between the 914 mm and 1220 mm headers should be valved shut.

This arrangement would provide duplicate raw water connections from the low lift station to the process units, thereby providing complete security of raw water supply.

Within the existing low lift pumping station, we recommend improvements to the lighting and ventilation, and also that the equipment removal monorail in the high lift station be extended through to this station to enable low lift pumps to be removed for maintenance or repair, if necessary.

6.5 LOW LIFT SYSTEM - SUMMARY RECOMMENDATIONS

We recommend that additional raw water piping be added between the low lift station and treatment modules to provide security of supply. This work should be completed in 1983.

We further recommend the installation of a second raw water screen and new raw water pump in the existing low lift station by 1990.

SECTION 7

WATER TREATMENT FACILITIES

7.1 GENERAL

The original treatment process included a combination flocculation/sedimentation unit - a proprietary unit known as an Accelator - and filtration. Plant operating experience determined that the effective rating (approx. 27 000 m^3/d) of this unit was substantially lower than required, and that chemical quantities necessary for it to operate effectively were very high when low turbidity water was being treated.

The Accelator was therefore taken out of service and new flocculation tanks constructed so that the plant would operate in the 'direct filtration' mode.

At present the treatment facilities comprise flocculation to agglomerate the suspended coagulated particles present in the raw water, and filtration to reduce the suspended solids concentration in the water to below a level equivalent to a turbidity of 1.0 NTU.

As described in Section 4 of this report, the particular raw water quality characteristics at Oakville are such that direct filtration is a highly effective process for most of the year, but that on occasions the extremely turbid raw water cannot be treated in this manner and a 'full treatment' configuration is required.

7.2 FLOCCULATION

The recently constructed flocculation tanks comprise two three-celled units each with a capacity of 54 500 m^3/d at a 16.4 minute retention time for a total flocculation capacity of 109 000 m^3/d. Each cell is equipped with a variable speed drive impeller to promote flocculation, designed to provide a tapered, or reduced energy, floccuation mode.

In order to meet the projected maximum plant capacity requirements, additional flocculation tanks will be required to give a total capacity of 169 700 m^3/d.

Based on plant performance, except in times of very high raw water turbidities, the performance of the existing units, as would be expected, is satisfactory. The design basis for the new flocculation tanks would be the same as the present design, except that we propose hydraulic mixing within the tanks, rather than mechanical mixing.

It will be necessary to make an internal modification to the existing tanks at the outlet end to allow flocculated water to flow by gravity to the new sedimentation tank, thus allowing the existing outlet conduit to serve as a connection between the sedimentation tank and existing filters.

7.3 FILTERS

The existing filters comprise eight dual media conventional filters which have recently been reconstructed internally so that their present rated capacity is:

- four filters - 58 m^2 each at 11.7 m/hr = 65 100 m^3/d gross
- four filters - 58 m^2 each at 9.8 m/hr = 54 500 m^3/d gross

TOTAL CAPACITY - GROSS = 119 600 m^3/d

The net capacity of these filters, assuming a thirty minute wash (out-of-service) period per day, is thus 117 000 m^3/d.

In order to meet the ultimate plant capacity requirements, additional filtration capacity will be required to give a total capacity of 169 700 m^3/d.

Based on present plant performance, the existing filters function satisfactorily, as would be expected. No alterations are proposed to these filters.

To meet the projected maximum plant requirements, it is proposed to construct two additional filters each approximately 115 square metres area, designed to operate at a 10 m/hr rate giving an effective additional capacity, allowing for out of service during backwashing, of 54 000 m^3/d.

The design for the new filters would be essentially the same as the existing filters in terms of media arrangement, monitoring, and controls.

7.4 SEDIMENTATION

At present there are no operating facilities for sedimentation, since the use of the existing Accelator (having a capacity of less than 30 000 m^3/d) has been discontinued. To provide the treatment process recommended in Section 4, therefore, new sedimentation facilities are required.

Two alternatives are available for providing the necessary sedimentation capacity - either the existing Accelator could be brought back into service and additional facilities constructed to provide the balance of sedimentation capacity; or an entirely new sedimentation tank could be constructed for the whole capacity required.

The cost of construction of the full capacity settling tank and refurbishing the Accelator to act as a waste treatment tank is substantially less than the construction of a slightly smaller settling tank and the construction of a new tank for wastewater treatment. We recommend therefore that entirely new sedimentation facilities be constructed, and the existing Accelator be used for wastewater treatment.

A major factor to be considered in an evaluation of any treatment process is the relationship between the greatest demands on the process for production, i.e. quantity of water to be produced, and the greatest demands on the process, i.e. high turbidity or other process variable.

Table 7-1 opposite indicates, for the years 1977 to 1981, the ratio of average day flow in any month to average day flow for the whole year.

Table 3-1 indicates, for the same time period, the normal raw water turbidity recorded, and where some values greatly exceeded the normal range for that month, those values are also indicated in that table.

A comparison of the data presented in these two tables indicates the relationship between high turbidities and high flows. Typically, the turbidities high enough to preclude adequate plant performance without sedimentation occur during periods of the year when the demand for water production is lower than average.

For this reason, the design of the sedimentation facility obviously need not be based on treating flows at maximum day rates since there are no occurrences of high turbidities during the part of the year when maximum day demands have to be met. A 'high-rate' design based on maximum day flow is, therefore, appropriate.

We would recommend that the basis for design of the sedimentation tank should permit overflow rates approaching the maximum recommended by the Ministry of the Environment Design Guidelines when operating at maximum day rate. We believe that an overflow rate of 4.8 m/hr would be appropriate at maximum day conditions, thus requiring an effective area of approximately 1450 square metres. This rate can only be achieved when using inclined tube settlers installed within the tank. Since tube settlers can be easily installed in a tank specifically designed for them, it is more economical to construct a smaller tank at this time and, as required, install settling tubes in the tank when the flows are such that the performance without tubes deteriorates, than to construct additional tankage at that time.

In the absence of the tube settlers, reasonable performance of the settling tank could be expected at rates up to approximately 1.5 m/hr during cold water conditions. The effective capacity of the tank alone is, therefore, 52 000 m^3/d. This capacity is expected to be exceeded in 1993, by which time tube settlers should be added to uprate the tank.

With regard to cleaning settled sludge from the tank, we believe that the provision of automatic sludge removal equipment is not justifiable. For most of the year, it will only be necessary to add small doses of coagulant to treat the low turbidity water. Under this condition, the flocculated material will be sufficiently light that only small amounts will settle in the tank. During high turbidity periods, sludge will accumulate at a substantially higher rate. The high turbidity periods, however, are of short duration and tank performance is unlikely to be adversely affected by sludge deposition during these periods.

7.5 WATER TREATMENT FACILITIES - SUMMARY RECOMMENDATIONS

We recommend the construction of a sedimentation tank in 1983, adjacent to the existing flocculation tanks.

We recommend the addition of two sets of flocculation tanks by 1990, adjacent to the recommended additional filters.

We recommend the addition of two new filters, and settling tubes in the sedimentation tank by 1993.

SECTION 8

HIGH LIFT PUMPING STATION

8.1 PURPOSE

The high lift pumping station draws treated water from the clearwell storage and reservoir, and pumps this water to the distribution system at an appropriate pressure.

8.2 CAPACITY REQUIRED

The required output from this station must be sufficient to meet the maximum day requirement of 161 000 m^3/d, all other in-system demands being met from system storage. The Insurer's Advisory Organization, with which we concur, recommends that such capacity be available from the pumping station with the largest single unit out of service.

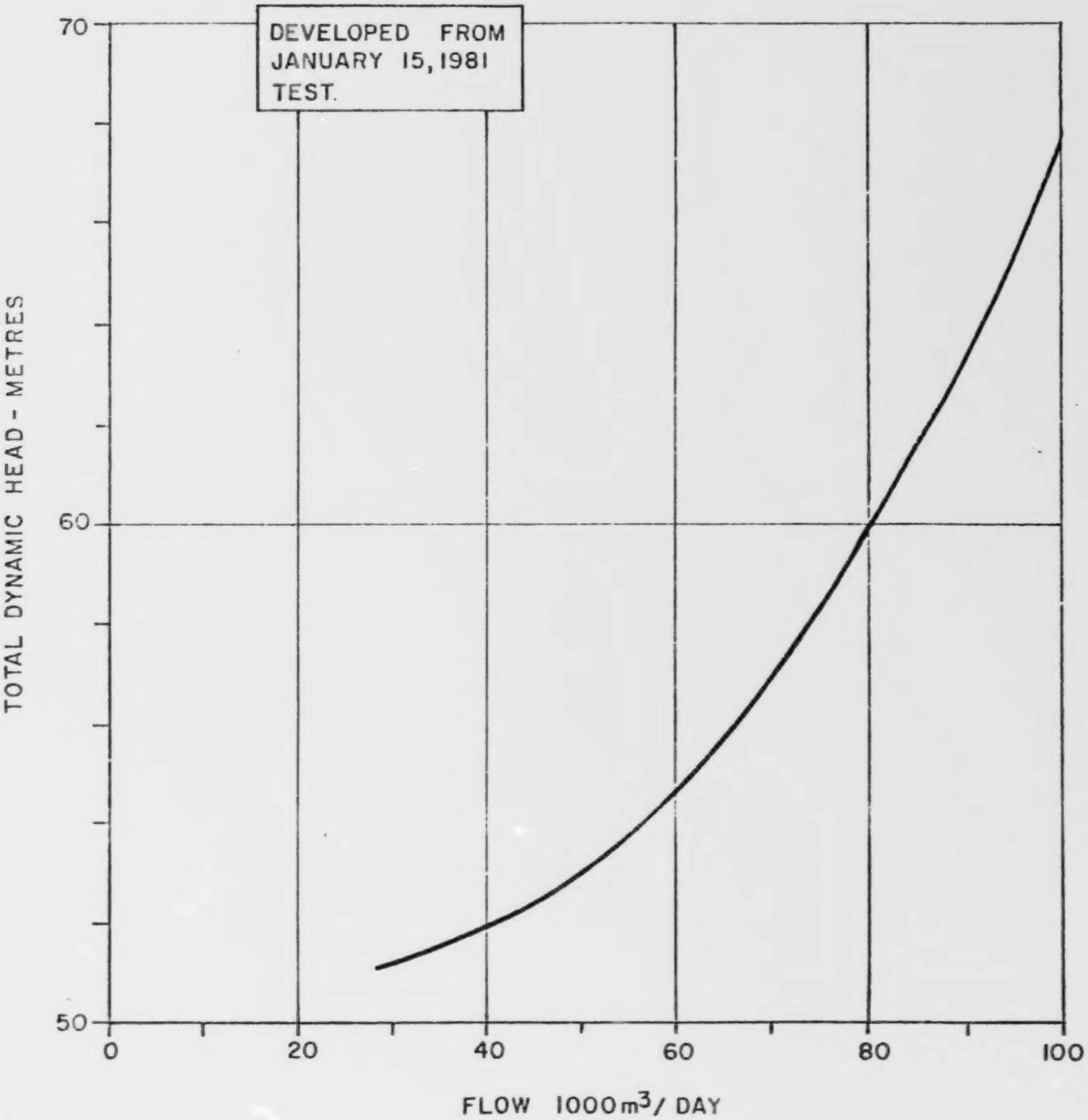
8.3 EXISTING EQUIPMENT

Our report on the Town of Oakville Water Distribution System, Appendix 4, describes in detail the testing performed in 1981 with the existing equipment. Appendix 2 of this report reproduces Appendix 4 of the Distribution System Report for convenient reference. The results of those tests are summarized in the Table 8-1 below:

TABLE 8-1: EXISTING HIGH LIFT PUMP CAPACITIES

PUMP #	MAKE	INSTALLED	CAPACITY AT 70 m HEAD
1	Ingersoll-Rand	1977	34 000 m^3/d
2	DeLaval	1953	7 000 m^3/d
3	DeLaval	1961	22 500 m^3/d
4	DeLaval	1971	29 500 m^3/d

SYSTEM HEAD CURVE
HIGH LIFT PUMPING STATION



Simcoe

FIGURE 2

The total installed capacity is, therefore, 93 000 m^3/d at 70 m head; the firm capacity is 59 000 m^3/d at 70 m head, and, as determined from the system head curve (Figure 2 opposite), the firm capacity of the existing equipment is 82 000 m^3/d at 60 metres total head.

The existing equipment, therefore, is not sufficient to meet the projected maximum demand, and additional and/or replacement equipment will be needed.

8.4 HYDRAULIC ANALYSIS

A major consideration in the selection of equipment for future use is that the distribution system be protected from extreme hydraulic transients - either low or high pressure.

A surge analysis was conducted for the total load rejection condition (power failure with all pumps running) using a pumping capacity equal to maximum day output. The analysis assumed the addition of a 900 mm feedermain from the treatment plant to the Davis Road Booster Station. This watermain has been previously recommended for construction in 1992, and it is assumed that it will be installed prior to the ultimate flow condition being reached.

The analysis showed that no major upsurge would occur, but that an unacceptable downsurge to approximately -100 kPa would result. Further analysis indicated that the transient minimum pressure could be increased to approximately +110 kPa - an acceptable level - by the inclusion of a 34 m^3 capacity hydro-pneumatic tank connected to the discharge piping.

TABLE 8-2: PROPOSED HIGH LIFT PUMPS - CAPACITY m^3/d

	Existing	Stage 1*	Stage 2*
Pump #1 at 70 m	34 000	34 000	34 000
Pump #2 at 70 m	7 000	35 000	35 000
Pump #3 at 70 m	22 500	35 000	35 000
Pump #4 at 70 m	29 500	29 500	29 500
Pump #5 at 70 m	-	-	35 000
Pump #6 at 70 m	-	-	35 000
FIRM CAPACITY AT 70 m HEAD	59 000	98 500	168 500
FIRM CAPACITY AT SYSTEM HEAD	82 000	103 000	162 000
ADEQUATE UNTIL YEAR	1984	1991	ULTIMATE

*STAGE 1 is alterations within existing station

*STAGE 2 is construction of additional station

In the downtown Core Area there are many very old cast iron watermains which would probably fail in the event that substantial upsurges occurred on pump startup. To limit these upsurges to 70 kPa, which we believe will provide an adequate margin of safety for these watermains, it is recommended that in addition to the hydropneumatic tank, both high lift pumps not presently having both cushioned check valves to prevent slamming on power failure, and slow opening discharge valves to prevent surges on startup, be so equipped.

8.5 RECOMMENDED PUMPING UNITS

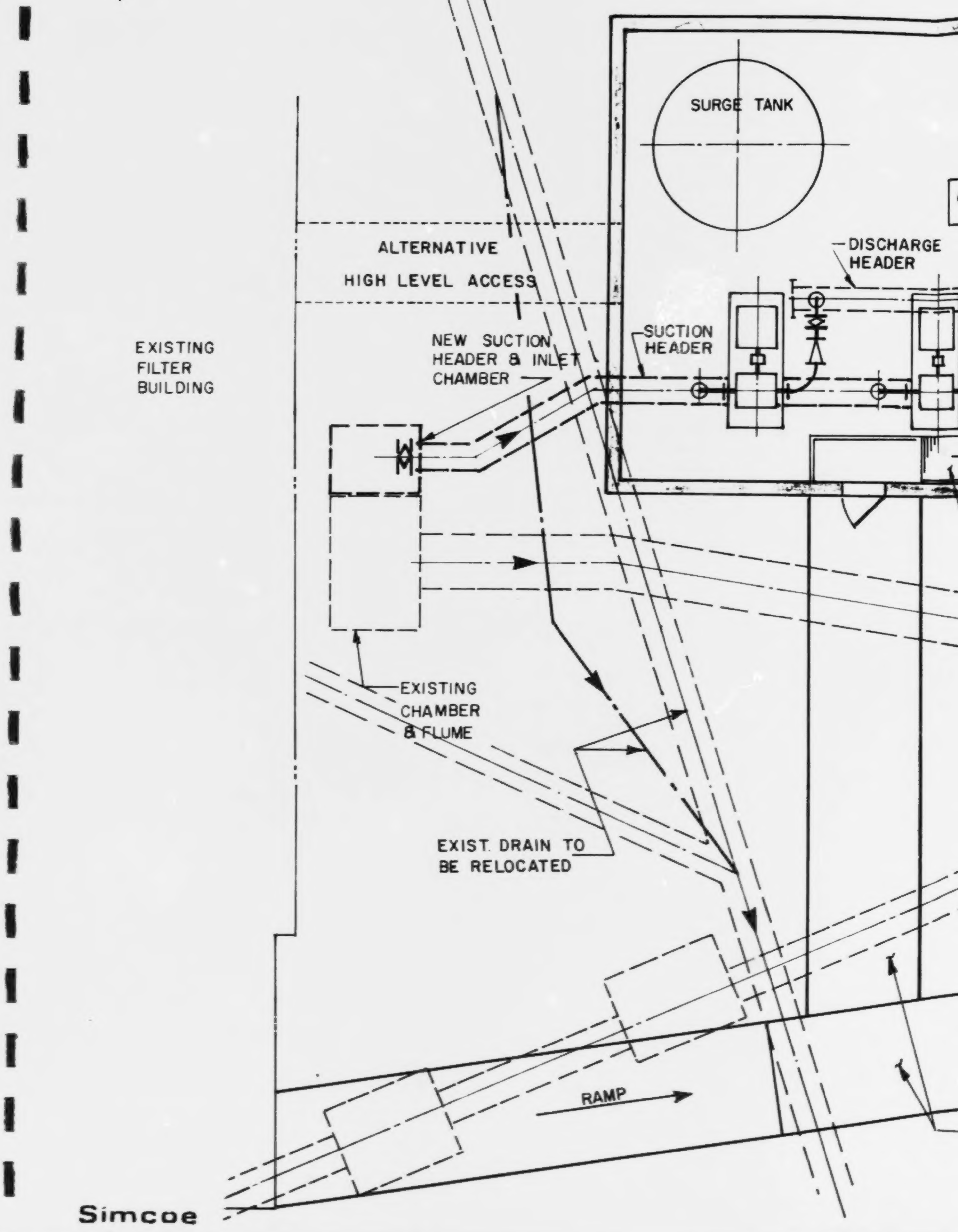
The existing equipment has sufficient firm capacity to meet demands until 1984. At that time it would be sufficient to replace the smallest existing unit with a much larger pump, thus deferring construction of a new building for additional pumps. As discussed in Appendix 4 of our report on the Water Distribution System, however, we would recommend replacement of the two smallest pumps (#2 and #3) with larger units.

As shown in Table 8-2, such replacements will provide sufficient firm capacity to meet demands until 1991. Since it is not possible to install any further pumps in the existing station, nor increase the size of these pumps beyond the 1991 capacity, it will be necessary to provide a new pumping station for further capacity beyond 1991. The proposed station arrangement is shown in Figure 3.

8.6 IMPROVEMENT TO EXISTING PUMPING STATION

In order to complete all work within this pumping station, certain other improvements are recommended. These include the

PROPOSED PUMPI


SURGE TANK
DISCHARGE HEADER
SUCTION HEADER
ALTERNATIVE
HIGH LEVEL ACCESS
NEW SUCTION HEADER & INLET CHAMBER
EXISTING FILTER BUILDING
EXISTING CHAMBER & FLUME
EXIST. DRAIN TO BE RELOCATED
RAMP
Simcoe

G STATION ARRANGEMENT

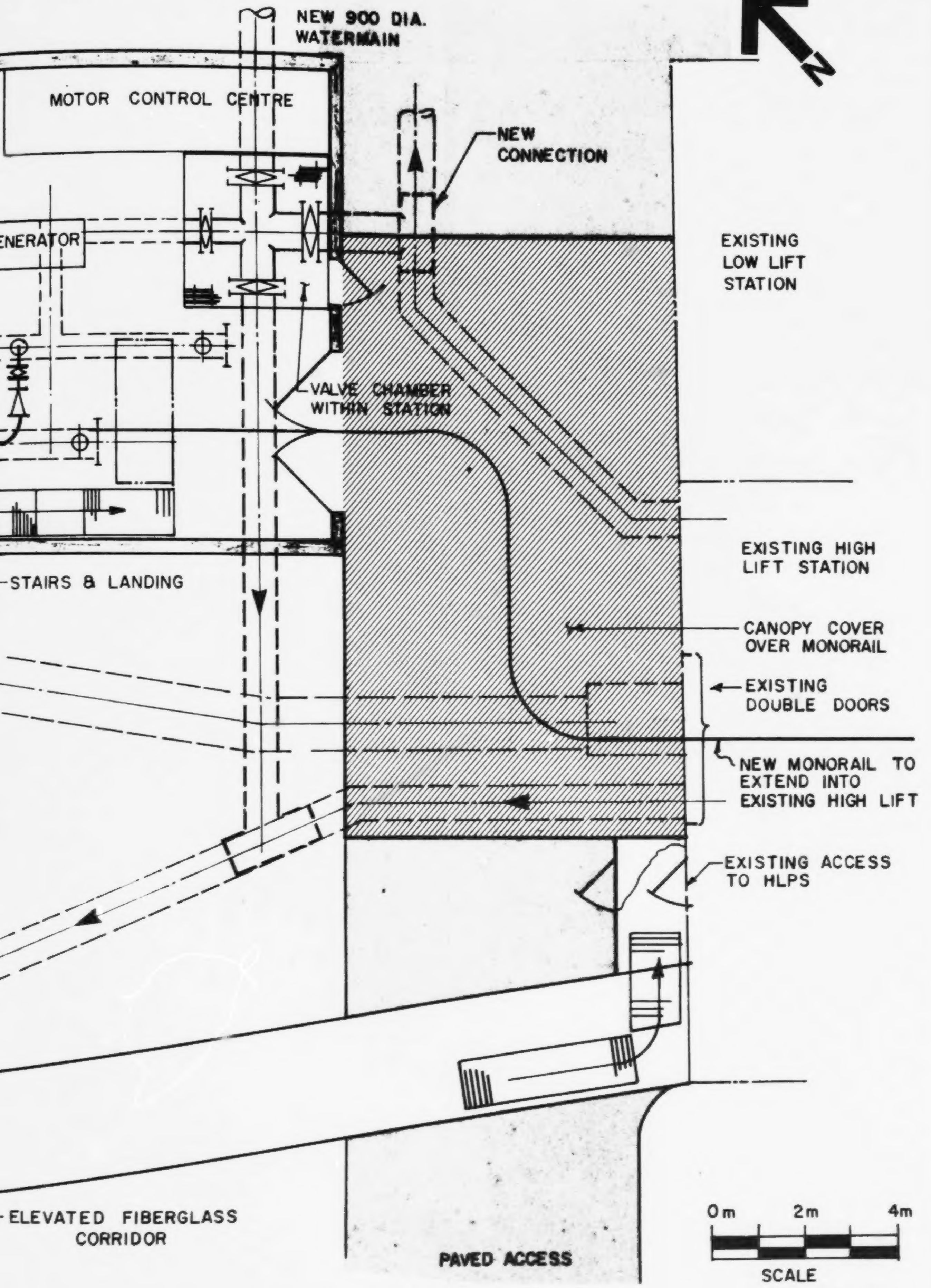

NEW 900 DIA. WATERMAIN
N
MOTOR CONTROL CENTRE
NEW CONNECTION
ENERATOR
EXISTING LOW LIFT STATION
VALVE CHAMBER WITHIN STATION
EXISTING HIGH LIFT STATION
STAIRS & LANDING
CANOPY COVER OVER MONORAIL
EXISTING DOUBLE DOORS
NEW MONORAIL TO EXTEND INTO EXISTING HIGH LIFT
EXISTING ACCESS TO HLPS
ELEVATED FIBERGLASS CORRIDOR
PAVED ACCESS
0m
2m
4m
SCALE

FIGURE 3

addition of automatic discharge valves on the two pumps which presently have manually operated valves, and alterations to enable the existing equipment to be more readily accessible for service and maintenance, viz. improving the existing lifting beams and crane arrangement within the station.

8.7 HIGH LIFT SYSTEM - SUMMARY RECOMMENDATIONS

We recommend that improvements be made within the existing pumping station to improve the general operation of this station. These improvements should be made in 1983.

We recommend the replacement of the two smaller pumps within this station with larger units at that time.

We further recommend that in 1991 a second high lift pumping station be built, complementing the existing station, and be equipped with two pumping units and a surge protection tank.

SECTION 9

CHEMICALS AND CHEMICAL HANDLING FACILITIES

9.1 GENERAL

The present chemical facilities comprise systems for storage and dispensing of chlorine for disinfection; liquid alum and polyelectrolyte for turbidity control; sodium chlorite/chlorine dioxide for disinfection and taste/odour control; and hydrofluosilicic acid for dental caries prevention.

The existing facilities are distributed throughout the operating areas of the treatment plant, as shown in Figure 4 following. The present situation is not desirable for the following reasons:

a) the most economical control system requires that all chemical systems be located in close proximity to each other;

b) chemical deliveries to a single location rather than several simplifies supervision of delivery;

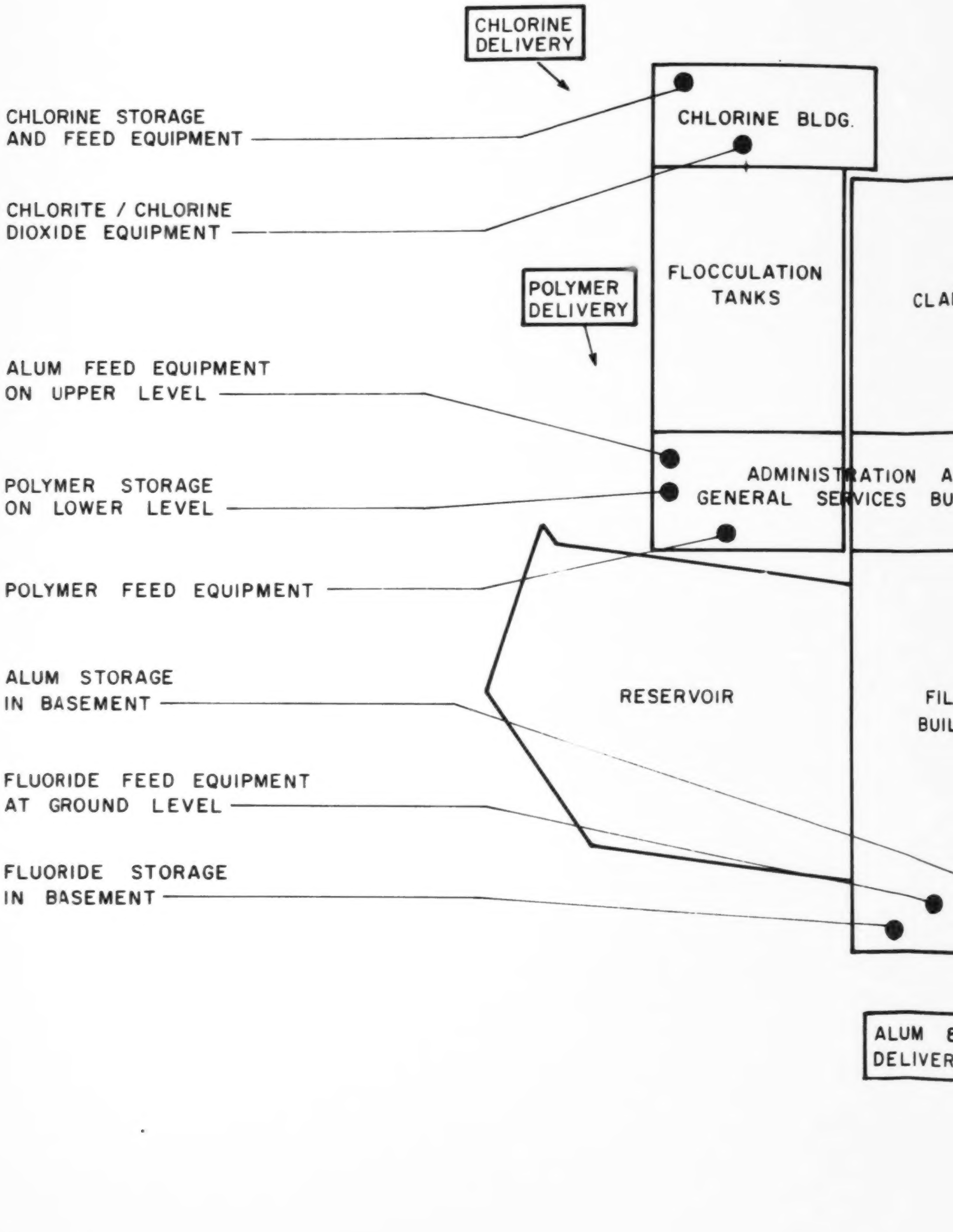
c) routine inspections of the chemical systems are more convenient when all systems are grouped together;

d) special safety systems or HVAC requirements can be more economically provided in a single area than for widely dispersed areas throughout the plant;

e) personnel hazards are reduced and containment of accidental spills is simplified by concentrating chemical storage into a single area;

f) in the event that manual operation of the chemical system is necessary, widely dispersed individual systems increase

EXISTING CHEMICAL SYSTEMS


CHLORINE DELIVERY
CHLORINE STORAGE AND FEED EQUIPMENT
CHLORINE BLDG.
CHLORITE / CHLORINE DIOXIDE EQUIPMENT
FLOCCULATION TANKS
POLYMER DELIVERY
CLA
ALUM FEED EQUIPMENT ON UPPER LEVEL
ADMINISTRATION A
GENERAL SERVICES BU
POLYMER STORAGE ON LOWER LEVEL
POLYMER FEED EQUIPMENT
RESERVOIR
ALUM STORAGE IN BASEMENT
FIL
BUIL
FLUORIDE FEED EQUIPMENT AT GROUND LEVEL
FLUORIDE STORAGE IN BASEMENT
ALUM &
DELIVER

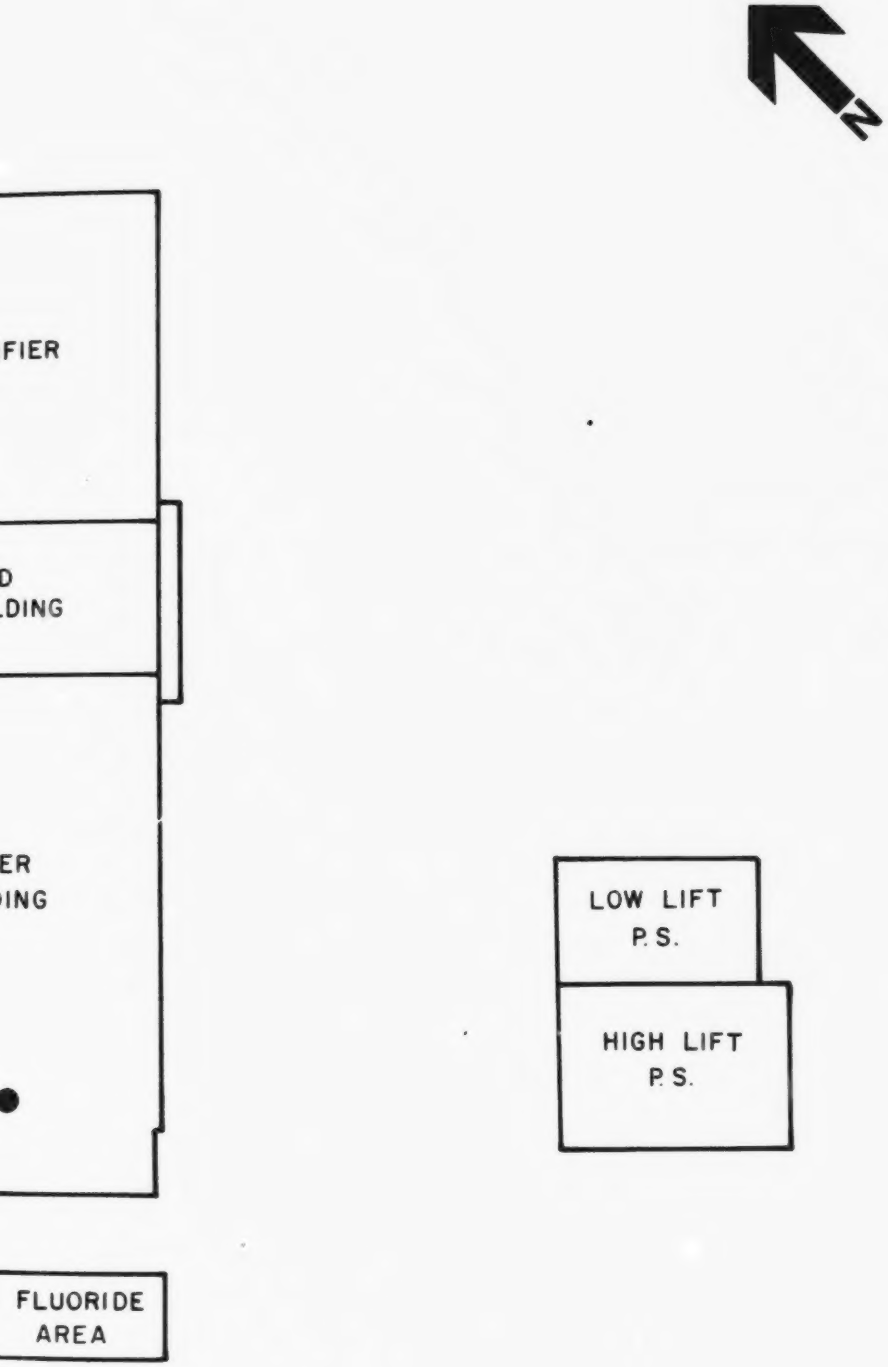

N
FIER
D
LDING
ER
DING
LOW LIFT
P.S.
HIGH LIFT
P.S.
FLUORIDE
AREA

FIGURE 4

manpower needs.

9.2 CHEMICAL SYSTEMS REQUIREMENTS

The basis for chemical storage and feed requirements should be consistent with the approach recommended by the MOE Guidelines for Design of Water Treatment Works. The guidelines may be summarized as requiring:

a) storage of sufficient chemical for the quantity of water treated in any month;

b) feed equipment, with back-up equipment, capable of feeding sufficient chemicals to treat a flow 20% higher than maximum day (to allow margin for short-term flows in excess of maximum day rates);

9.3 CHEMICAL SYSTEMS

9.3.1 Chlorine System

The existing chlorine system is a recently designed and constructed system without deficiencies, and no additional work is required to bring this system to meet the ultimate plant capacity.

9.3.2 Aluminum Sulphate System

The existing storage tank is not sufficiently large to provide the recommended storage for the ultimate plant capacity. The underground location is not suitable for a modern treatment plant, and a new storage tank should be provided elsewhere. The existing feed equipment is recently constructed and suitable for continued use.

9.3.3 Hydrofluosilicic Acid System

The existing storage tank is not sufficiently large to provide the recommended storage for the ultimate plant capacity. The underground location is potentially hazardous to operating staff and such a location is not suitable for a modern treatment plant, and new storage should be provided elsewhere. The existing feed equipment is suitable for re-use.

9.3.4 Sodium Chlorite/Chlorine Dioxide

This system has been recently reconstructed and located adjacent to the chlorine storage area. The present location and feed equipment are satisfactory, and the system is suitable for use without alteration to meet the utlimate plant capacity demands.

9.3.5 Polyelectrolyte System

This system was recently constructed to provide full backup for both liquid and powdered polyelectrolytes. Bulk storage of polymer is adjacent to the preparation and dispensing equipment.

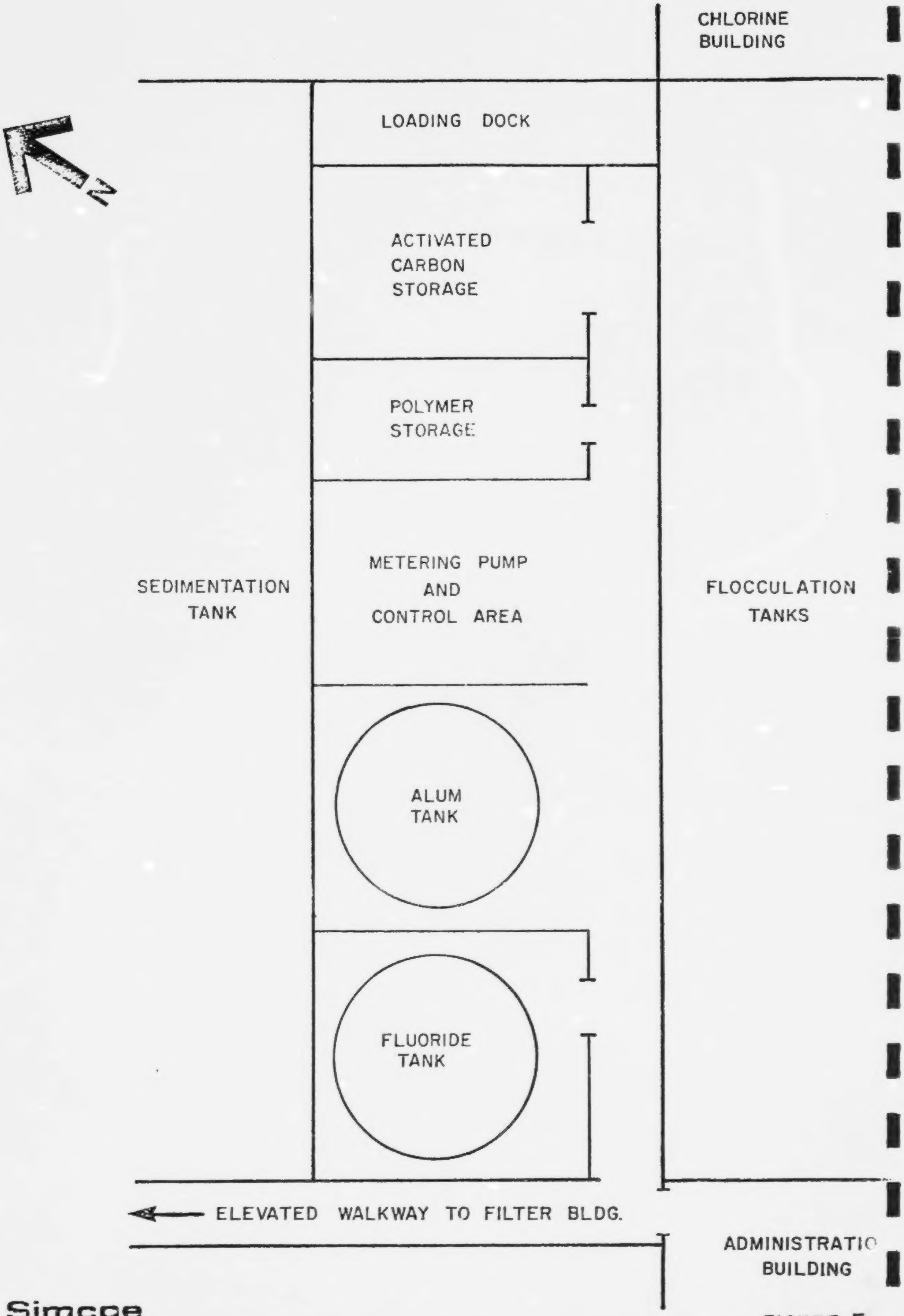
On the basis of plant performance since this system was installed, it appears safe to conclude that liquid polymers will continue to perform effectively, and be the sole type used in future.

It is recommended therefore that a simplified feed system be constructed and located with the other chemical systems, and that the present dry polymer feed systems be disconnected and 'mothballed', except that the metering pumps be relocated and re-used.

9.3.6 Activated Carbon System

As recommended in Section 4, provision should be made for

PROPOSED CHEMICAL BUILDING


CHLORINE BUILDING
LOADING DOCK
N
ACTIVATED CARBON STORAGE
POLYMER STORAGE
SEDIMENTATION TANK
METERING PUMP AND CONTROL AREA
FLOCCULATION TANKS
ALUM TANK
FLUORIDE TANK
ELEVATED WALKWAY TO FILTER BLDG.
ADMINISTRATIO
BUILDING

Simcoe

FIGURE 5

a powdered activated carbon system to be included in the chemical building.

The system should include a bulk dry storage silo for truckload deliveries; a dry feeder system transferring the powdered carbon to a slurry tank and metering pumps. The operation should be automated to minimize the need for manual intervention in the system.

9.4 NEW CHEMICAL BUILDING

To provide the systems discussed above, in a single area of the plant, requires the construction of a new building. It is recommended that this building be constructed above the new sedimentation tank, thus requiring no foundation or excavation costs, abutting the existing chlorine building.

This new building would then provide an integrated chemical system, simplifying routine inspection or manual operation. A single access road would allow for deliveries of all chemicals to be made from Wilson Street, and site security would be improved.

Figure 5 illustrates the recommended arrangement of the chemical building.

9.5 CHEMICAL SYSTEMS - SUMMARY RECOMMENDATIONS

We recommend that new centralized chemical storage and feed facilities be constructed in 1983, located adjacent to the existing chlorine storage area.

We recommend that new storage tanks be provided for aluminum sulphate and hydrofluosilicic acid, and that equipment be provided for storage and feeding of powdered activated carbon.

SECTION 10

CONTROL SYSTEM

10.1 EXISTING SYSTEM

The present control system for the Oakville treatment plant may be characterized as primarily a manual control system, in that most plant equipment is manually started, adjusted or controlled either at the equipment, or at the local control panels, such as filter control consoles. Major exceptions are the chemical feed systems which automatically adjust to changes in rate of the water being treated, and the production cycle of filtration where filter rate adjusts automatically to compensate for changes in level of the clearwell.

10.2 PROPOSED SYSTEM

To provide improved control over both the water treatment plant and its associated distribution system, and also to reduce the manpower requirements for routine operation of the plant, an automated control system was proposed for the Oakville plant, similar to that being installed at the Burlington plant.

The rationale and philosophy of such a system for Oakville was outlined in the James F. MacLaren report entitled 'Report on the Modernization of Supervisory Control' submitted in November, 1980. Subsequently the system described in that report was modified to more closely match the Burlington system, and is presently under design by Simcoe Engineering Limited.

The future system at the Oakville plant will be connected to the Burlington Treatment Plant by telephone line and will make use of the computing power and data storage of the Burlington system, which was designed to accommodate such a use. Figure 6 illustrates the 'system architecture' for the combined treatment plant systems, showing the proposed Oakville system in detail.

The system design and mode of operation is essentially identical to that for the Burlington Plant thus permitting the interchange of operating personnel between plants without retraining.

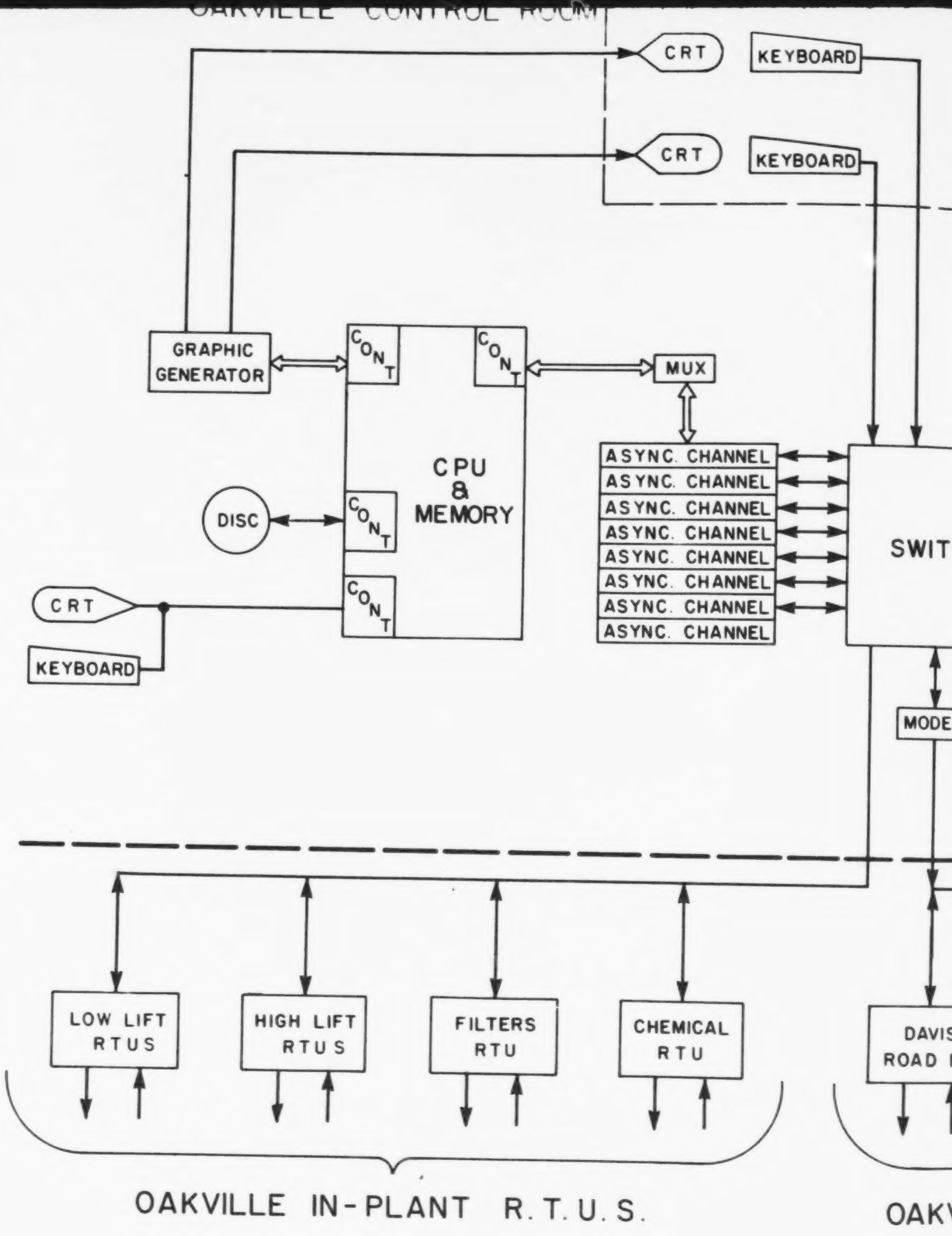
10.2.1 Central Control System - Burlington

The Burlington central control computers will continuously monitor the state of the Oakville plant through a telephone link between the plants. The Burlington equipment also provides data base storage and historical recording for both treatment plants and will be responsible for most management functions, such as report generation.

10.2.2 Central Control - Oakville

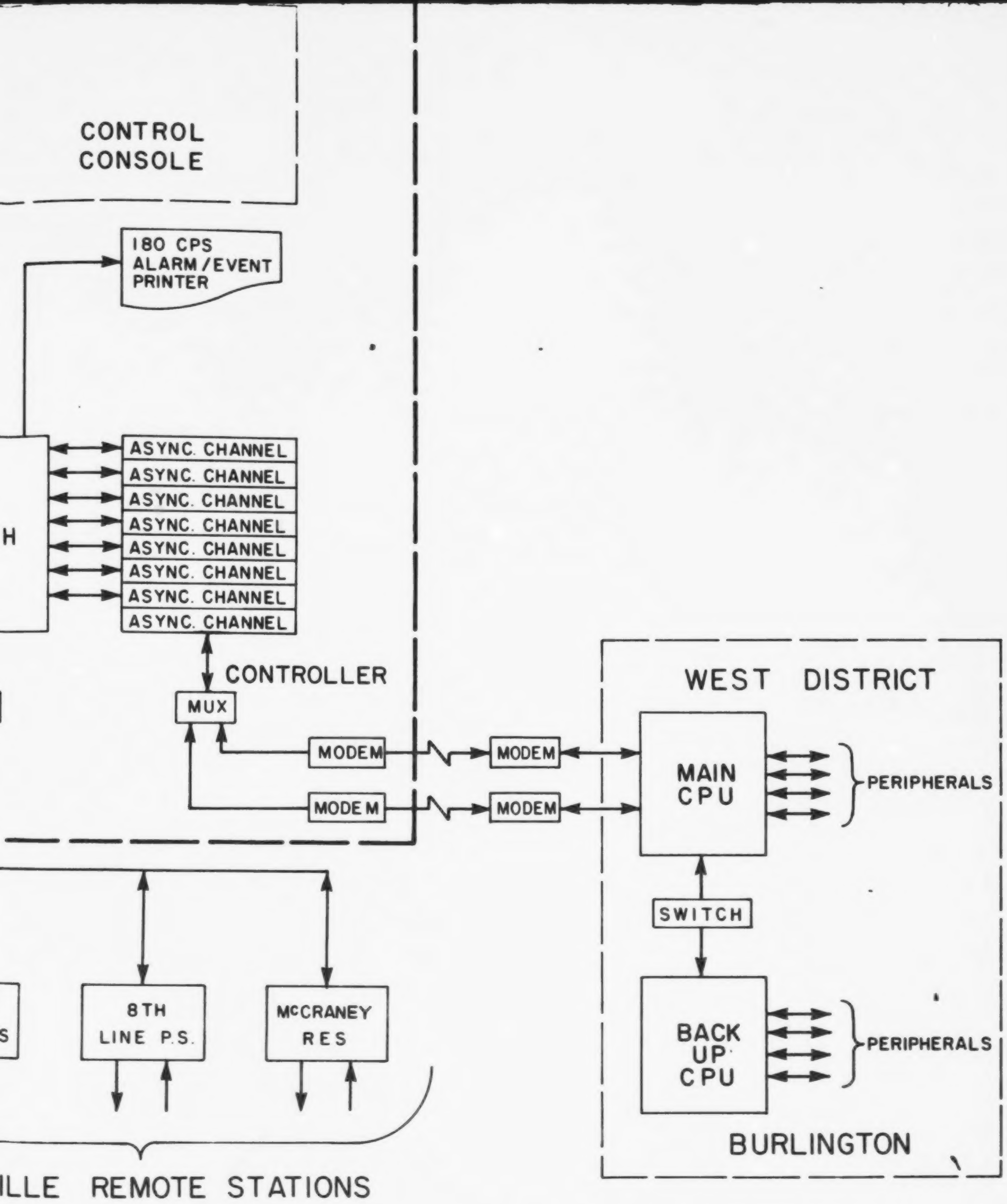
The central control system at the Oakville Plant will comprise a single mini-computer which will access the Burlington computers for historical data. A disc storage system will be incorporated to provide the colour graphic displays, since transmission of graphic data base via telephone line is slow, and graphic display generation from a remote data base could take several minutes.

Other equipment to be provided will include two 19-inch colour graphic display monitors, and a high speed printer for


OAKVILLE CONTROL ROOM
CRT
KEYBOARD
CRT
KEYBOARD
GRAPHIC GENERATOR
CONT
CONT
MUX
CPU & MEMORY
DISC
CONT
CONT
CRT
KEYBOARD
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
SWIT
MODE
LOW LIFT RTUS
HIGH LIFT RTUS
FILTERS RTU
CHEMICAL RTU
DAVIS
ROAD
OAKVILLE IN-PLANT R.T.U.S.
OAKV

PROPOSED CONFIGURATION FO

Simcoe


CONTROL CONSOLE
180 CPS ALARM / EVENT PRINTER
H
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
ASYNC. CHANNEL
CONTROLLER
MUX
MODEM
MODEM
MODEM
MODEM
WEST DISTRICT
MAIN CPU
PERIPHERALS
SWITCH
BACK UP CPU
PERIPHERALS
BURLINGTON
S
8TH LINE P.S.
McCRANEY RES
ILLE REMOTE STATIONS

R OAKVILLE / BURLINGTON WATER SYSTEM

FIGURE 6

alarm/event reporting. Two custom keyboards based on the Burlington keyboards will be provided.

This equipment will be included in a single console in the new control room. It is recommended that, in addition to this equipment in central control, a micro-computer and monochrome CRT be provided in the water plant engineer's office so that the Burlington computer can be interrogated for system information independent of the operations of the Oakville plant.

10.2.3 Remote Terminal Units

Commands from the Oakville computer are passed to the remote terminal units located at key areas of the plant - pumping stations, chemical area, and filter buildings. Commands are then directed from the remote terminal units (RTU's) to the appropriate equipment.

Field generated data, i.e. analog signals from transmitters, digital signals from status points (such as alarm levels), are transmitted from the field or local devices to the RTU.

The central computer routinely polls the RTU's to determine changes of state of digital outputs, i.e. change of pump status, alarms reached, etc., and obtain updated analog values from level, pressure, etc. transmitters.

The remote terminal unit is also capable of operating as a local control centre since it can be used to start or stop equipment and display status or values of process variables and alarms. The units are 'intelligent' in that they incorporate certain preprogrammed functions so that in the event of a communications failure between RTU and central control, the unit will maintain its current state or fail-safe to predetermined conditions.

10.2.4 Local Control Units

The existing filter control consoles will remain in service to provide a further level of control security. Other local control panels will be modified to act as data collection points, thus minimizing the overall cost of implementing the system.

10.2.5 Remote Pumping Stations

The remote pumping stations in the Oakville distribution system will also be modified to include RTU's to control/monitor the equipment at each station. Functionally these units will be identical to those provided at all of the pumping stations in Burlington.

10.3 REQUIRED IMPROVEMENTS AT OAKVILLE PLANT

A substantial number of alterations and improvements will be required to provide all of the necessary devices which produce the analog and digital outputs to be monitored by the central system. Typically this includes the provision of additional limit switches on valves to indicate their position, level transmitters on chemical storage tanks, etc.

Under a present construction contract the existing filter building will have many of these modifications made, including the provision of the filter building remote terminal unit.

10.4 APPLICATIONS SOFTWARE

Applications and operating software was developed for the Burlington system at a cost of $250 000. To a very large extent the same software is applicable directly to the Oakville plant. The cost of the Oakville system should be reduced by the 're-use' of this software.

10.5 CONTROL SYSTEM - SUMMARY RECOMMENDATIONS

We recommend that in 1983 a supervisory control system be implemented in the Oakville plant and distribution system as described in this section.

We recommend that the control system be connected to the Burlington Water Plant main computer, using identical software to that already in place in Burlington.

SECTION 11

POWER SYSTEM

11.1 EXISTING SYSTEM

The plant is presently fed power at 4160 volts from an existing transformer station located immediately north of the plant boundary on Kerr Street, with an alternate feed of limited capacity from the Margaret Street municipal substation. Both of these stations are now the property of Oakville Hydro.

The peak load of the present plant, as measured during April 1981, was 1536 kVA. Up until some time in 1986 Oakville Hydro is prepared to continue supplying power from the Kerr Street transformer station up to a maximum of 2000 kVA, as noted in their letter of April 6th, 1982 (Appendix 3). The capacity available from the alternate Margaret Street transformer station is limited to only approximately 950 kVA , and thus is inadequate to meet the present peak demand of the water treatment plant alone.

The existing electrical distribution in the plant is divided into a 4160 volt system supplying the larger pump motors, and a 600 volt system supplying some smaller pump and compressor motors, electrical unit heaters, overhead crane, chemical feed equipment, etc., plus a number of 600:120/240 V, 1 phase and 600:120/208 V, 3 phase distribution transformers for lighting and other building services. The present 600 volt system is supplied by two 500 kVA 4160:600 V transformers via individual 600 volt buses with a tie breaker for interconnecting in the event of transformer failure.

TABLE 11-1: ESTIMATED POWER DEMAND

PRESENT

ITEM	kVA**	MAXIMUM* HP	MAXIMUM* kVA
LOW LIFT PUMP, 200 HP	180	400	360
LOW LIFT PUMP, 200 HP	180		
LOW LIFT PUMP, 100 HP	90		
LOW LIFT PUMP, 100 HP	90		
HIGH LIFT PUMP, 500 HP	450	1100	990
HIGH LIFT PUMP, 250 HP	215		
HIGH LIFT PUMP, 350 HP	315		
HIGH LIFT PUMP, 550 HP	450		
WASHWATER PUMP, 100 HP	90	100	90
WASHWATER PUMP, 100 HP	90		
COMPRESSOR/BLOWER, 50 HP	50	50	50
MISCELLANEOUS, 100 HP	100	50	50
		TOTAL	1540

FUTURE

ITEM	kVA**	MAXIMUM* HP	MAXIMUM* kVA
LOW LIFT PUMP, 200 HP	180	600	540
LOW LIFT PUMP, 200 HP	180		
LOW LIFT PUMP, 100 HP	90		
LOW LIFT PUMP, 100 HP	90		
LOW LIFT PUMP, 200 HP	180		
HIGH LIFT PUMP, 500 HP	450	2500	2250
HIGH LIFT PUMP, 500 HP	450		
HIGH LIFT PUMP, 500 HP	450		
HIGH LIFT PUMP, 500 HP	450		
HIGH LIFT PUMP, 500 HP	450		
HIGH LIFT PUMP, 500 HP	450		
WASHWATER PUMP, 100 HP	90	200	180
WASHWATER PUMP, 100 HP	90		
WASHWATER PUMP, 200 HP	180	200	180
WASHWATER PUMP, 200 HP	180	200	180
COMPRESSOR/BLOWER, 50 HP	50	50	50
MISCELLANEOUS, 200 HP	200	100	100
		TOTAL	3120

*MAXIMUM TOTAL CONNECTED SIMULTANEOUSLY
**ASSUMING P.F. CORRECTED TO 0.90

11.2 FUTURE POWER REQUIREMENTS

In the recommended expansions practically all of the additional load will be at 4160 volts, so that the existing 600 volt transformers should be able to carry the 600 volt loads for the foreseeable future. However, additional capacity will be required to supply the 4160 volt system. Table 11-1 shows estimates of the various items contributing to the present and future power demand of the plant.

From the present peak demand of approximately 1540 kVA, the demand is projected to increase to an ultimate total in excess of 3000 kVA. For a limited time the 2000 kVA maximum from the Kerr Street transformer station imposed by Oakville Hydro should be sufficient for carrying the plant load. However, in the event of a failure of the Kerr Street station, the peak demand cannot be supplied from the alternate Margaret Street station even at the present level of operation.

In addition, Oakville Hydro has advised that the full capacity of the Kerr Street station will be required to supply the needs of their other customers in the Kerr Street and Lakeshore Road area when the planned extensions of their underground distribution system are undertaken in 1986. At that time they will undertake to supply the Oakville Water Treatment Plant with 27.6 kV primary voltage only, and will require that the plant include its own transformer station. Regardless of any other considerations, therefore, a new transformer station must be included as part of the plant extensions.

11.3 POWER SUPPLY ALTERNATIVES

There are two alternatives to consider:

1. The transformer station might be designed to include initially two 2000 kVA transformers fed in parallel from a single 27.6 kV hydro primary feeder. For standby power, a diesel generator set of at least 2000 kVA capacity would be required to meet average demand loads.
2. If two alternate 27.6 kV hydro primary feeders were available, the station could be designed to include two 2000 kVA transformers initially as in alternative 1, with the two 4160 volt secondary buses connected to a normally-open tie switch. We believe that the security offered by the two separate hydro feeders would preclude the need for a diesel generator standby set. In the rare event of the failure of one of the transformers, it could be cut out of service, and by closing the tie switch the plant could be operated up to the 2000 kVA capacity of the remaining transformer. As the plant demand increased, a third 2000 kVA transformer and tie switch could be added to the system, thus providing emergency capacity of 4000 kVA if one transformer should fail.

Oakville Hydro have stated that when the present Kerr Street station becomes unavailable to the Region, in about 1986, they should have in place a loop primary feed system from two separate Ontario Hydro lines. A sub-loop from this system could be made available to supply the water plant transformer station, thus providing the security of Alternative 2, above.

PROPOSED POWER SUPPLY SYSTEM

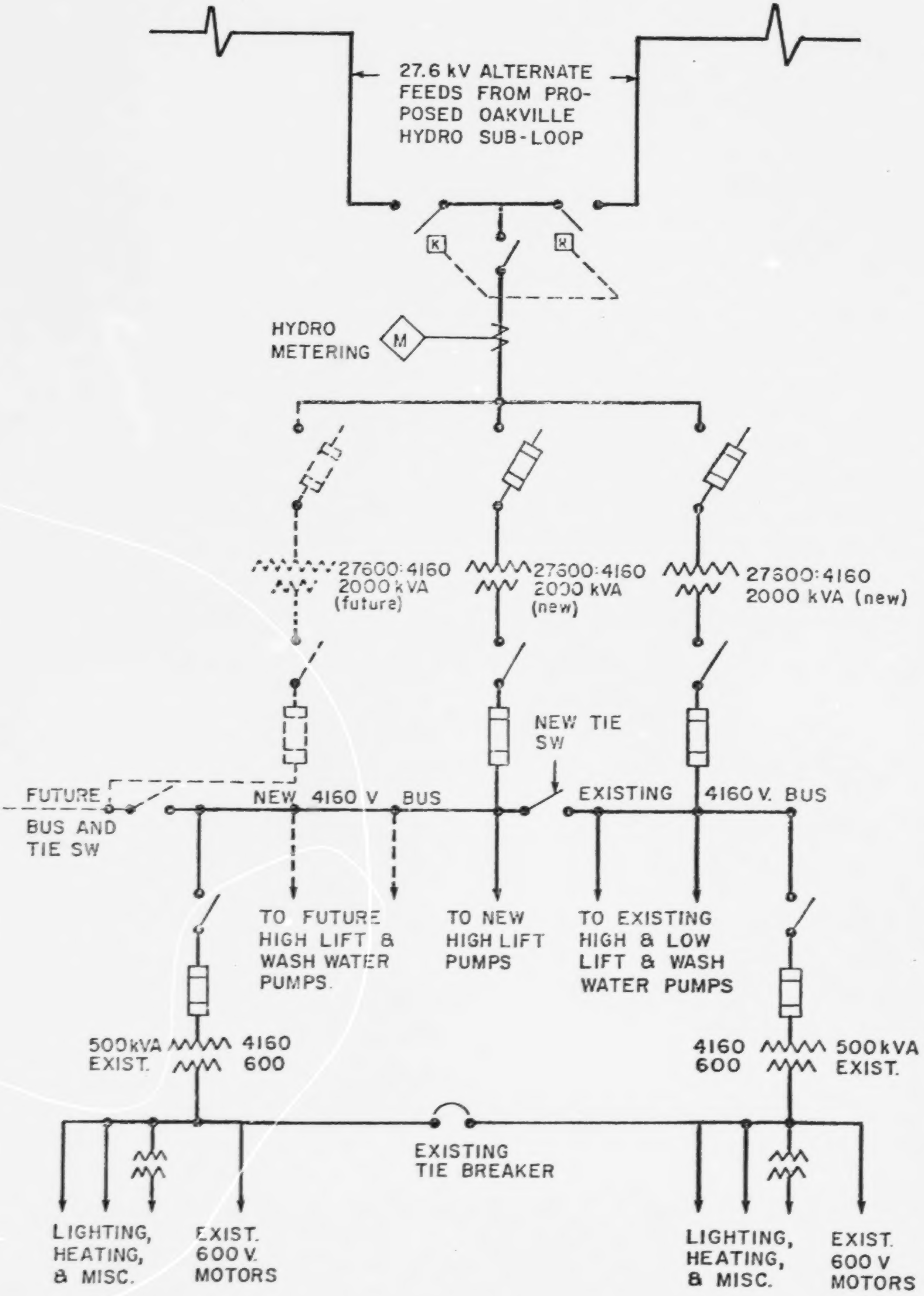

27.6 kV ALTERNATE FEEDS FROM PROPOSED OAKVILLE HYDRO SUB-LOOP
K
K
HYDRO METERING
M
27600:4160 2000 kVA (future)
27600:4160 2000 kVA (new)
27600:4160 2000 kVA (new)
NEW TIE SW
FUTURE BUS AND TIE SW
NEW 4160 V BUS
EXISTING 4160 V. BUS
TO FUTURE HIGH LIFT & WASH WATER PUMPS.
TO NEW HIGH LIFT PUMPS
TO EXISTING HIGH & LOW LIFT & WASH WATER PUMPS
500kVA EXIST.
4160
600
4160
600
500kVA EXIST.
EXISTING TIE BREAKER
LIGHTING, HEATING, & MISC.
EXIST. 600 V. MOTORS
LIGHTING, HEATING, & MISC.
EXIST. 600 V MOTORS

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FIGURE 7

The cost of providing emergency standby power, at the 2000 kVA loads, would far exceed the cost of two alternate hydro primary feeds including loop-capacity cable in underground ducts and necessary switchgear. We recommend the provision of the dual 2000 kVA transformers without standby power generator, as shown in the single line diagram, Figure 7. Appendix 4 provides a system reliability analysis for this configuration.

Oakville Hydro have indicated that, in the event of an outage on one feeder, their personnel could reach the transformer station and switch over to the alternate feeder within one hour. In order to supply essential process control and monitoring, and some building services during such an outage, we recommend that a small emergency diesel generator (say 100 kW) supplying power at 600 volts be provided. There is more than sufficient treated water storage within the distribution system reservoirs to maintain a supply during such outages.

In addition, water can be supplied to the Oakville system Zone 1 from Burlington through an existing 150 mm watermain on Lakeshore Road, and when the Rebecca Street 500 mm interconnection is made (scheduled for 1984) an additional supply from Burlington Zone 1 will be available.

11.4 POWER SYSTEM - SUMMARY RECOMMENDATIONS

We recommend that a new transformer station be constructed in 1983, with an initial capacity of 4000 kVA, to be increased by 1993 to 6000 kVA.

We also recommend the provision of a small standby diesel generator capable of providing power for lighting, controls and instrumentation, and other minor uses, during any power interruption.

SECTION 12

AUXILIARY FACILITIES

12.1 GENERAL

In addition to the treatment and production related facilities discussed in earlier sections, certain additional facilities are required at the Oakville treatment plant, namely improvements to site access and internal roadways, wastewater treatment, an improved administration and operations building, and architectural improvements/renovations to the existing structures.

12.2 SITE ACCESS

At present there are two access roads onto the site. On the West side, access is available to the existing filter building and pumping stations; on the East side access is available for deliveries of chlorine, polyelectrolyte, and miscellaneous supplies. There is no direct access to the plant main entrance.

It is recommended that improvements be made to on-site roadways as shown in the drawing R1. Access to the chlorine building and new chemical building will be improved to allow convenient delivery for large semi-trailers normally used for bulk chemical deliveries. The existing west access will be extended past the low lift pumping station towards the main plant entrance, and incorporate parking spaces for plant personnel and visitor vehicles. A new access will be constructed to serve the washwater/wastewater pumping area and equipment supply delivery area at the north end of the administration building.

WASTEWATER TREATMENT SYSTEM

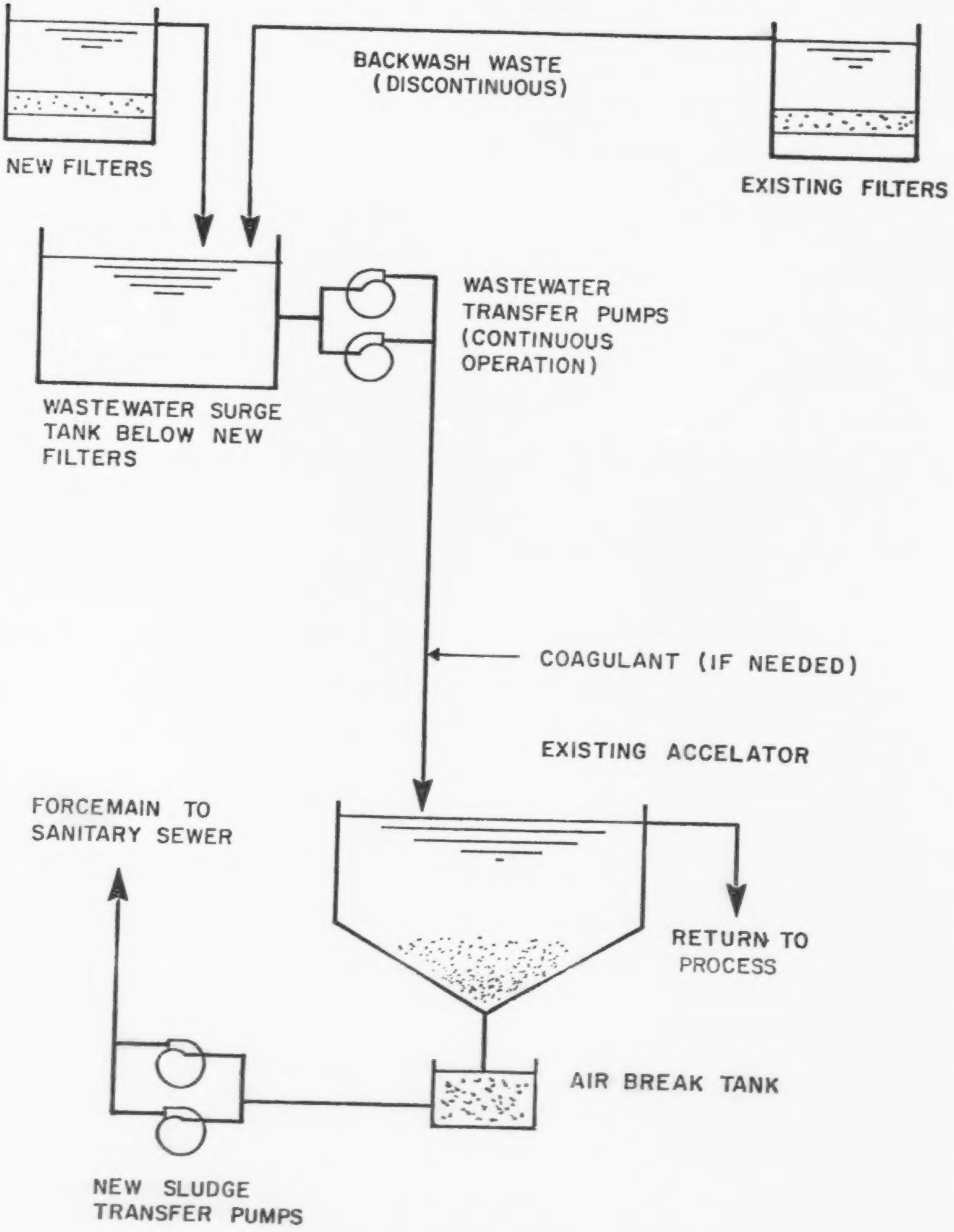
NEW FILTERS
BACKWASH WASTE (DISCONTINUOUS)
EXISTING FILTERS
WASTEWATER TRANSFER PUMPS (CONTINUOUS OPERATION)
WASTEWATER SURGE TANK BELOW NEW FILTERS
COAGULANT (IF NEEDED)
EXISTING ACCELATOR
FORCEMAIN TO SANITARY SEWER
RETURN TO PROCESS
AIR BREAK TANK
NEW SLUDGE TRANSFER PUMPS

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FIGURE 8

12.3 WASTEWATER TREATMENT

In keeping with the present policies of the Ministry of the Environment it will be necessary to provide treatment of the process wastes generated. At present these wastes are discharged to the storm sewer system.

It is recommended that the treatment comprise settling of wastes in the existing Accelator with the overflow returned to the process, and underflow drawn off and pumped to the sanitary sewer system. Figure 8 shows the method of treatment.

12.3.1 Wastewater Holding Tank

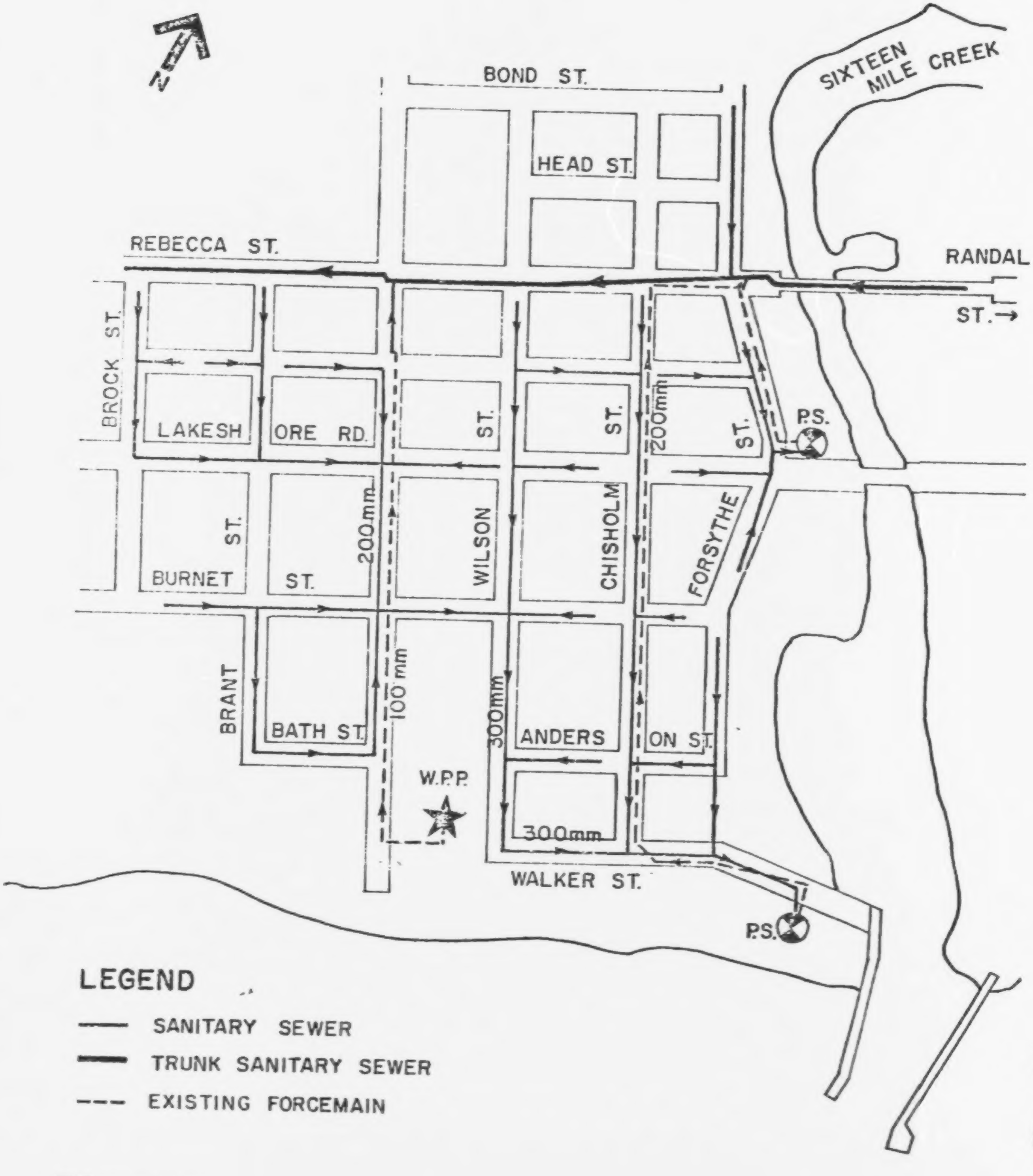
The most economical design for the wastewater treatment will be for it to operate continuously at a uniform rate. However, backwashing filters is a discontinous operation. In order that the system operate at a uniform rate, it is necessary to provide a surge or holding tank to receive incoming wastewater at a high rate. The maximum daily inflow to this tank will be equivalent to the volume produced from backwashing all filters, estimated at 6500 cubic metres.

Since this washwater volume only arises under circumstances where all filters must be washed during a single 24-hour period, and, as described below, full standby for transfer pumps is proposed, it is not essential that the holding tank capacity equal the maximum wastewater volume. A tank capacity of 4500 cubic metres is recommended.

12.3.2 Wastewater Transfer

Adjacent to the holding tank, two wastewater transfer pumps

OAKVILLE W.P.P. AREA SEWERAGE SYSTEM


N
BOND ST.
SIXTEEN MILE CREEK
HEAD ST.
REBECCA ST.
RANDAL ST. →
BROCK ST.
LAKESHORE RD.
WILSON ST.
CHISHOLM ST.
200mm
FORSYTHE ST.
P.S.
BURNET ST.
200mm
BRANT
BATH ST.
100 mm
300mm
ANDERSON ST.
W.P.P.
300mm
WALKER ST.
P.S.
LEGEND
—— SANITARY SEWER
▬▬ TRUNK SANITARY SEWER
- - - EXISTING FORCEMAIN

Simcoe

FIGURE 9

will be provided, each of 6500 m^3/d capacity, connected from the holding tank to the exisitng Accelator. It is proposed that a connection be made from the polymer feed system to this line so that the settling characteristics of the backwash wastes can be enhanced if necessary.

12.3.3 Wastewater Settling

The existing Accelator will be used as a sedimentation tank, i.e. without the present internal mechanisms in use, where clarified wastewater of suitable quality will overflow, and settled sludges will be drawn from the bottom of the unit.

12.3.4 Effluent Discharges

The Accelator overflow should be of sufficiently high quality that it can be returned to the process via the sedimentation tank. Sludge underflow should be discharged to the sanitary sewerage system. Provision should be made to discharge Accelator overflow to the existing storm sewer.

Figure 9 shows the present sanitary sewerage system in the vicinity of the Oakville plant. A recently installed 100 mm forcemain connects directly from the treatment plant to the Rebecca Street trunk sewer. It is anticipated that there will be sufficient flow at all times in the trunk sewer to adequately transport the water treatment plant sludges without settling out in the sewer.

The present sludge transfer pumps do not develop sufficient head to overcome the static and dynamic losses which will arise when the 100 mm forcemain is used for sludge transfer. It will,

therefore, be necessary to provide new sludge transfer pumps for this purpose.

12.3.5 System Operation

The operation of the wastewater system should be fully automatic without the need for operator action. Wastewater will be pumped from the waste holding tank at a continuous rate unless either the water level in the Accelator is too high, or level in the waste holding tank too low. If necessary, as determined by the wastewater treatment efficiency, polymer solution will be added to the pumped wastewater.

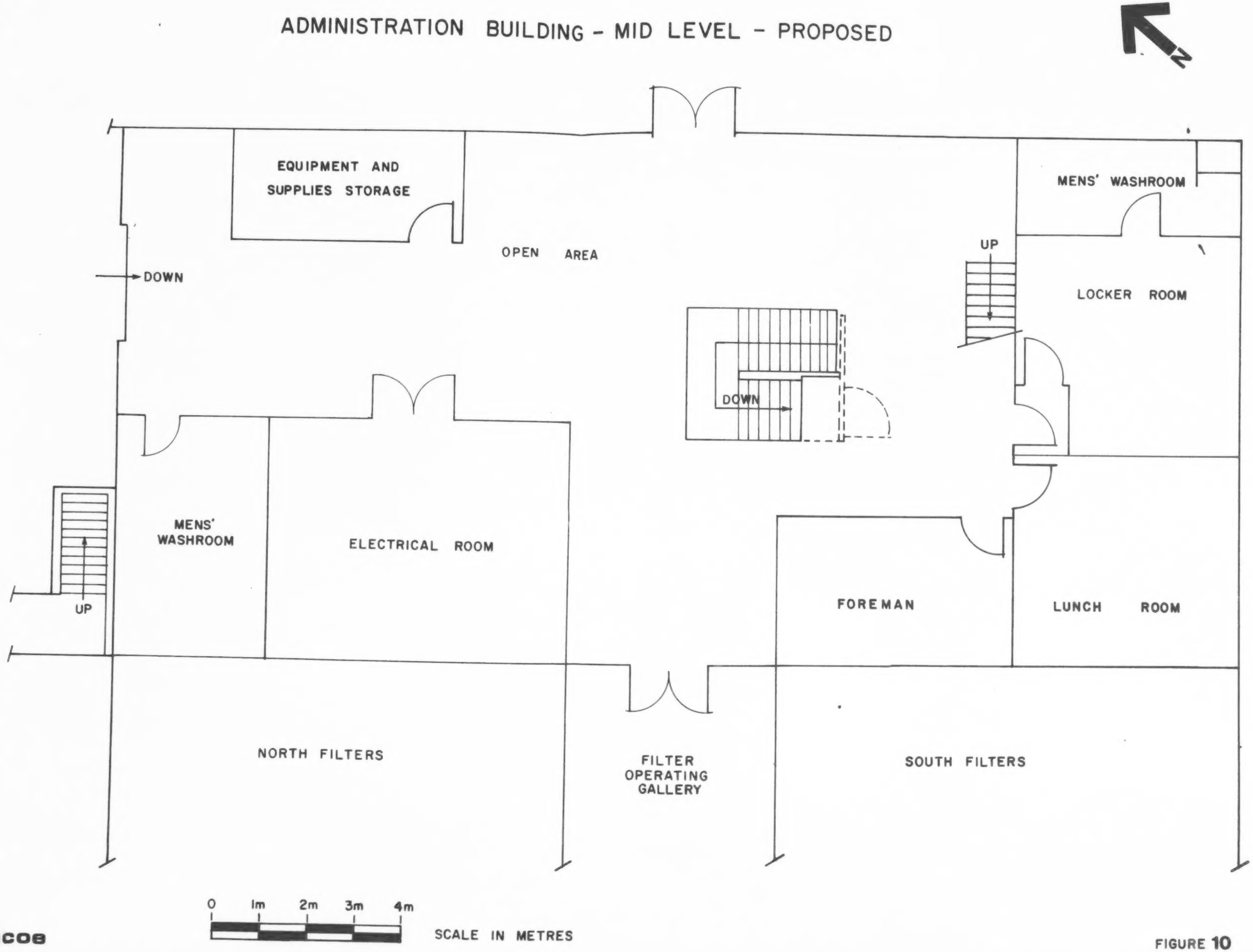
The underflow discharge system will be initiated by a 24-hour timer system, typically set to actuate for 1-2 hours/day. The underflow valve will open and remain open as long as the 'air-break' tank is not full. Sludge pumps will operate as long as the underflow quality indicates sludge is being pumped (typically by density meter). Once all sludge has been discharged, the system will shut down.

12.4 ADMINISTRATION AND OPERATIONS BUILDING

In order to provide a proper control room for the process control computer to be installed at the Oakville plant, new construction is required. In addition, the operating personnel facilities are distributed throughout a wide area of the plant and which is inconvenient. Laboratory facilities and pilot plant functions are dispersed through the plant and inadequate space is available.

Since the present upper level of the Administration Building is vacant and used only for general purpose storage, it is

ADMINISTRATION BUILDING - MID LEVEL - PROPOSED


N
EQUIPMENT AND SUPPLIES STORAGE
OPEN AREA
DOWN
MENS' WASHROOM
UP
LOCKER ROOM
DOWN
MENS' WASHROOM
ELECTRICAL ROOM
UP
FOREMAN
LUNCH ROOM
NORTH FILTERS
FILTER OPERATING GALLERY
SOUTH FILTERS
0
1m
2m
3m
4m
SCALE IN METRES

Simcoe

FIGURE 10

ADMINISTRATION BUILDING - UPPER LEVEL - PROPOSED

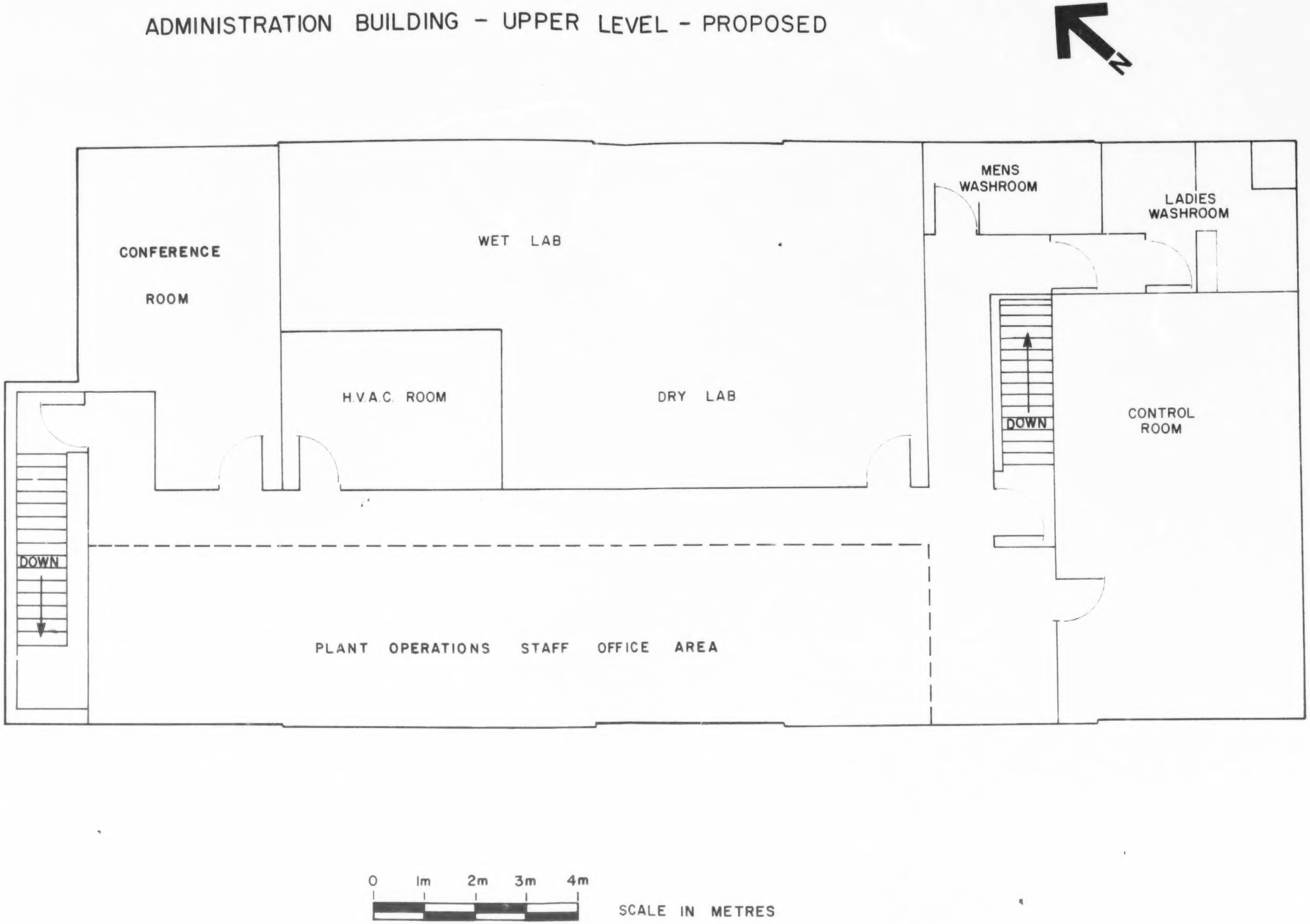

N
CONFERENCE ROOM
WET LAB
MENS WASHROOM
LADIES WASHROOM
H.V.A.C. ROOM
DRY LAB
DOWN
CONTROL ROOM
DOWN
PLANT OPERATIONS STAFF OFFICE AREA
0
1m
2m
3m
4m
SCALE IN METRES

FIGURE 11

recommended that the building be modified internally as shown in figures 10 and 11 following.

The arrangements shown will provide adequate facilities for the process control computer, a large dual purpose laboratory area enabling the present laboratory and pilot plant functions to be consolidated, and improved personnel facilities.

12.5 GENERAL IMPROVEMENTS

The overall appearance of the treatment plant buildings, as discussed in previous reports, presents an unbalanced appearance not in keeping with the present public uses of the plant site and immediate surroundings.

It is recommended that landscaping be provided on the south and west sides of the site, to complement the existing landscaping both within the site and in adjacent areas, and to be consistent with the public uses of the area.

It is also recommended that the existing filter and administration buildings be modified externally to provide a pleasant and cohesive appearance, and also to improve the energy efficiency of these buildings by replacement of existing single pane windows with modern units.

12.6 AUXILIARY FACILITIES - SUMMARY RECOMMENDATION

We recommend the construction of new site access roadways, and modification of existing roadways be made in 1983 to provide improved access and servicing of the treatment plant.

We recommend the construction in 1983 of a filter backwash waste holding tank, together with modifications to the existing Accelator to provide a complete wastewater treatment system compliant with Ministry of the Environment requirements.

We recommend internal modifications to the existing administration building in 1983 to provide a central control room, and improved laboratory and personnel facilities space.

We recommend external modifications to the exisiting structures for aesthetic improvement, and additional landscaping.

SECTION 13

PLANT LAYOUT

13.1 INTRODUCTION

As discussed in the previous sections, there is a need to construct the following process components:

- flocculation and sedimentation tanks
- filters
- chemical building
- high lift pumping station
- wastewater surge tank

While it is theoretically possible to locate these facilities anywhere within the site, practical considerations dictate that certain restraints be applied to the location of each component. These are:

- hydraulic considerations, and the desirability for gravity flow through the plant;
- adjacent locations for consecutive stages of the process;
- ease of hydraulic connection of new and existing components with the same duty;
- convenience of operation and maintenance accessibility.

The following sections indicate the rationale for the site development plan shown as Drawing R1 (inside back cover), and the recommended ultimate arrangement as shown in Figure 12 following.

13.2 SEDIMENTATION TANK

The sedimentation tank is recommended to be located immediately adjacent to the existing flocculation tanks, northward, for ease of connection to the flocculation tanks. This location also provides for a convenient central chemical storage area to be constructed immediately above the sedimentation tank.

13.3 FLOCCULATION TANKS

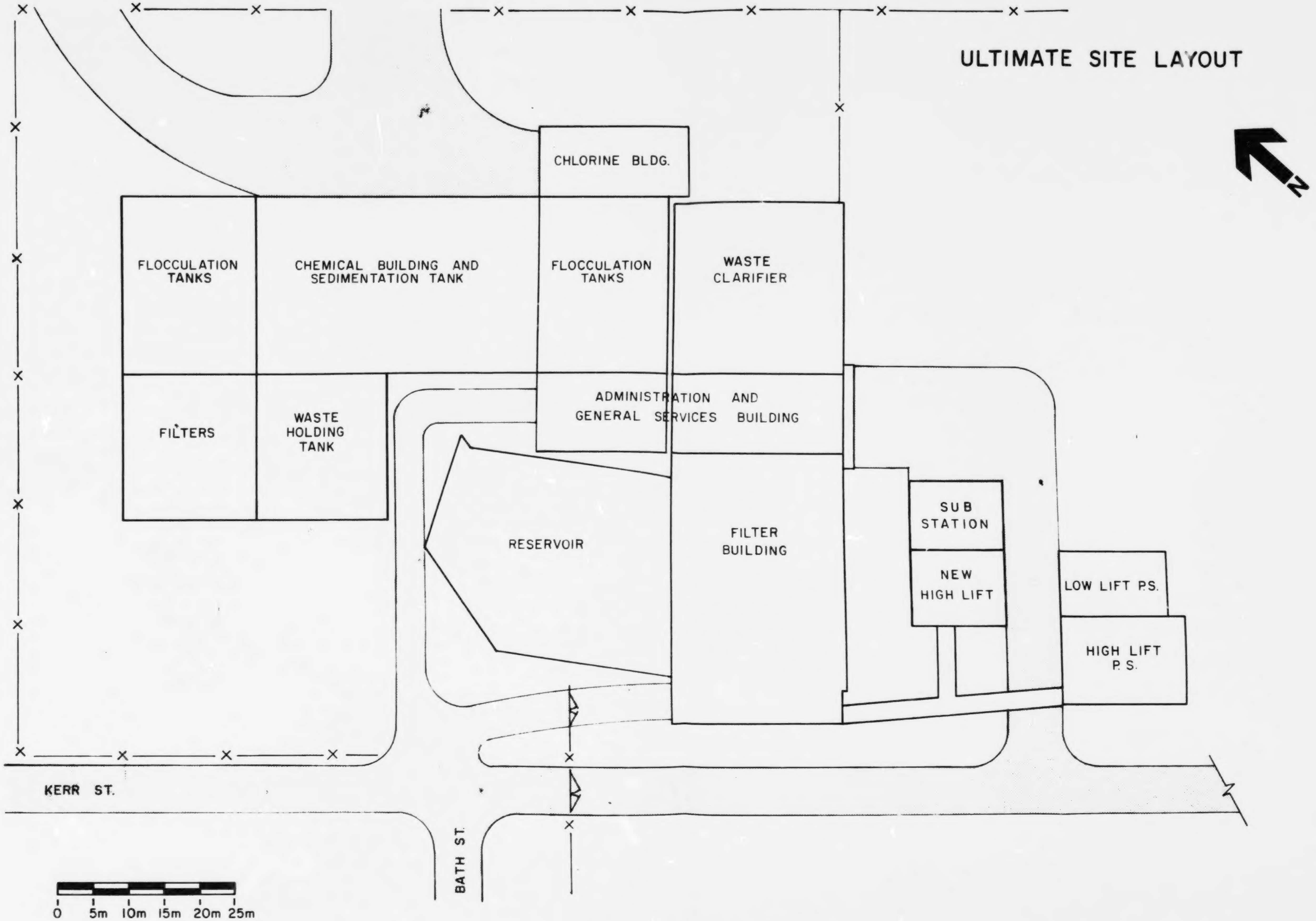
The present flocculation capacity is adequate for some years to come. The new flocculation tanks would normally be constructed immediately adjacent to the existing tanks, but in order to avoid the difficulties of constructing new tanks (at some time in the future) between the existing flocculation tanks and the then existing sedimentation tank, it is recommended that new flocculation tanks be constructed at the north end of the sedimentation tank.

13.4 FILTERS

Previous reports have indicated the potential for constructing new filters above the existing reservoir. On that basis a number of steel piles were driven between the reservoir and the polymer storage area at the time the 'pre-treatment' works were constructed. It was previously proposed that new filters be constructed from the piles, cantilevered out from the reservoir to the north, and with some additional support columns provided within the existing reservoir.

The advantages of construction at this location would be that the reservoir could serve as a clearwell below the new filters, thus reducing the net construction cost; that servicing and access to the filters would be simplified since they would abut the existing filters; and that none of the existing free space on the site would

ULTIMATE SITE LAYOUT


N
CHLORINE BLDG.
FLOCCULATION TANKS
CHEMICAL BUILDING AND SEDIMENTATION TANK
FLOCCULATION TANKS
WASTE CLARIFIER
FILTERS
WASTE HOLDING TANK
ADMINISTRATION AND GENERAL SERVICES BUILDING
RESERVOIR
FILTER BUILDING
SUB STATION
NEW HIGH LIFT
LOW LIFT P.S.
HIGH LIFT P.S.
KERR ST.
BATH ST.
0 5m 10m 15m 20m 25m

Simcoe

FIGURE 12

be used for filters, allowing space for reservoirs. The disadvantages of this location are that it would be necessary to remove the existing reservoir roof since there is insufficient space above the roof to construct filters with the same operating water level; and that the existing reservoir would be out of service for about nine months during the filter construction period.

Since we believe that no additional reservoir storage on site is required, the potential space saving advantage is nullified. We recommend that the filters be constructed away from the existing reservoir (which can thus remain in service at all times) adjacent to the sedimentation tank. Below the new filters, we recommend that in lieu of the conventional practice of constructing a clearwell, a washwater surge tank be constructed to take wastewater from both existing and new filters.

As the washwater surge tank constructed at a lower elevation than the filters is required in any event, the cost for its construction would be incurred wherever it was built.

By combining the new filters with the new washwater surge tank, and using the existing reservoir as the 'clearwell' for the new filters, the largest potential benefit of the originally recommended location is obtained without incurring the loss from service of the reservoir during construction.

Construction costs for new filters built away from the reservoir area would be lower since easier access is available, and we would expect that the added costs for hydraulic connections from the new filters to the reservoir would be no greater than the cost of removing the existing reservoir roof and filter construction in a less accessible area.

13.5 CHEMICAL BUILDING

We recommend that the new chemical building be constructed immediately above the sedimentation tank, adjacent to the present chlorine storage and feed areas. This will provide for a unified chemical storage area, with single access for chemical deliveries, thus improving both plant operations and security.

13.6 HIGH LIFT PUMPING STATION

We recommend that the new high lift pumping station be constructed between the existing station and existing filter building. It is possible for this new pumping station to be located at any part of the plant site where a positive hydraulic grade would exist from the finished water storage to the pumping station. However, in practical terms, it is more convenient to locate the new station adjacent to both the existing station and filter building. The advantages of this location are:

- convenient access for personnel from existing structures for supervision and maintenance
- convenient utility servicing
- low cost of connection from clearwell to pump suction header
- easy access for connection to existing discharge headers, and new watermains

13.7 WASTEWATER SURGE TANK

The location of the backwash wastewater surge tank is only constrained by the need to have gravity flow of wastewater from the filters to this tank. It can thus be located in any area of the site.

We recommend that this tank be constructed adjacent to the sedimentation tank in such a way that the future filters can be built immediately above it, thus reducing the overall cost of the two items.

13.8 SUBSTATION

We recommend that the new transformer substation be constructed adjacent to the new high lift pumping station, immediately opposite the existing low lift and high lift pumping station, since these process units contain almost all of the high power demand units in the treatment plant. The costs for high voltage distribution is thereby minimized. In addition, this location is more secure than most other alternatives, since it is overlooked by the control room and other personnel areas, thus improving supervision.

SECTION 14

ESTIMATED COSTS AND TIMING OF CONSTRUCTION

14.1 LIFE EXPECTANCY OF EXISTING FACILITIES

The timing of the various stages of construction is dependent on the probable dates when existing facilities are unable to meet the system requirements. Based on the demands projected in the Distribution System Study, and the capacity analyses detailed in earlier sections of this report, Table 14-1 following, was prepared to illustrate the timing needs for various extensions or improvements to the Oakville Water Treatment Plant.

14.2 STAGING OF CONSTRUCTION

In accordance with generally accepted practice, the construction of the works required to bring the plant to the recommended capacity should be divided into various stages to construct the facilities no earlier than necessary.

On this basis we recommend that the work be conducted in the following stages generally as shown in Figure 13 following.

14.2.1 Phase 1 Construction

This phase will include all works required either immediately or in the near future. The work will include:

- sedimentation tank
- chemical building
- administration building improvements
- pump replacement in high lift station, and station improvements
- low lift system improvements

TABLE 14-1: REQUIRED CONSTRUCTION DATES BASED ON PROJECTED SYSTEM DEMANDS

ITEM	COMMENTS
INTAKES	Capacity adequate for projected maximum demand. No improvement necessary.
LOW LIFT PUMPS	Capacity 109 000 m^3/d adequate until 1990.
FLOCCULATION	Capacity 109 000 m^3/d adequate until 1990.
SEDIMENTATION	Required immediately. Tube settlers required by 1993.
FILTRATION	Capacity 117 000 m^3/d adequate until 1993.
HIGH LIFT PUMPS	Capacity 82 000 m^3/d adequate to 1984. Pump replacement adequate to 1991. New station required in 1991.
CHEMICAL BUILDING	Required immediately.
CONTROL SYSTEM	Required immediately.
POWER SYSTEM	Replacement prior to 1986. Additional capacity required in 1993
ADMINISTRATION BUILDING	Required immediately.
WASTEWATER TREATMENT	Required immediately.

- substation with 2 x 2000 kVA transformers
- wastewater treatment system
- control system (as a separate contract)

14.2.2 Phase 2 Construction

This phase will include extensions or additions required to cover medium-term requirements, and these works should be completed by 1990. The work will include:

- new low lift pump
- additional raw water screen
- flocculation tanks
- new high lift pumping station

14.2.3 Phase 3 Construction

This phase will complete the alterations and additions necessary to bring the plant capacity throughout up to the projected maximum requirements, and these works should be completed by 1993. The work will include:

- new filters
- tube settlers installed in the settling tank
- additional 2000 kVA transformer

14.3 ESTIMATED COSTS OF CONSTRUCTION

The estimated construction costs in this section are based on a March 1982 construction index (ENR 3812), and include normal allowances for engineering costs and contingencies, totalling 15 per cent. It has been assumed that no abnormal sub-surface conditions will be encountered. It is assumed that there will be no special restraints placed on the work to comply with requirements of the Environmental Assessment Act.

OAKVILLE W.P.P. EXTENSIONS - PROJECT TIMING

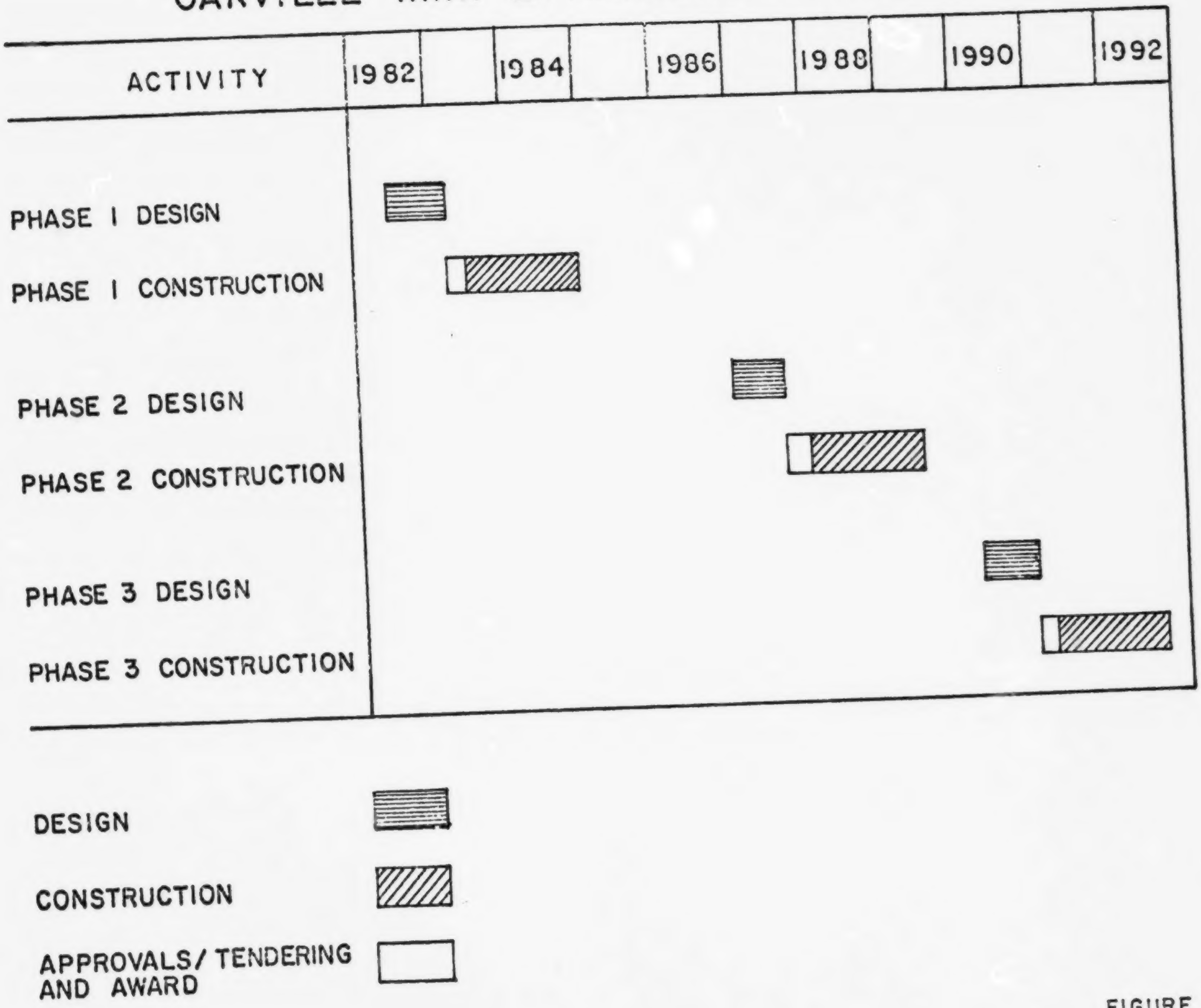


FIGURE 13

Phase 1 Construction - 1983/84

Sedimentation tank	$1 450 000
Chemical building	500 000
Administration building and site accesses	750 000
New high lift pumps and station improvements	400 000
New substation and feeders	400 000
Wastewater treatment	850 000
Low lift system improvements	200 000
Sub-total	$4 550 000
Control system computer contract	$ 750 000
Phase 1 estimated total cost	$5 300 000

Phase 2 Construction - 1988/89

Flocculation tanks	$2 100 000
New low lift pump and screen	150 000
New high lift pumping station	800 000
Phase 2 estimated total cost	$3 050 000

Phase 3 Construction - 1991/92

New filters	$1 800 000
Tube settlers	850 000
Substation extension	100 000
Phase 3 estimated total cost	$2 750 000

Summary - all construction phases

Phase 1	$5 300 000
Phase 2	3 050 000
Phase 3	2 750 000
Total estimated cost	$11 100 000

Appendix 1

Terms Of Reference

REGIONAL MUNICIPALITY OF HALTON

TERMS OF REFERENCE

FOR THE UPDATE ON STATUS OF OAKVILLE WATER TREATMENT PLANT IN PREPARATION FOR TENDERING OF WATER TREATMENT PLANT EXPANSION PROGRAM

In March 1977, JAMES F. MACLAREN LIMITED prepared the report "PRE-DESIGN REPORT ON THE EXPANSION OF THE OAKVILLE WATER TREATMENT PLANT". The study reviewed the plant's existing facilities and recommended an expansion programme to bring the capacity up to 109,000 cu. metre/day over a period of five (5) years.

Since that time, some of the more urgent recommendations contained in the report have been implemented. In continuation with the Oakville Water Treatment Plant expansion program, Subdivision Agreements have been entered into between the Region of Halton and several developers and have been formulated in the spirit of the Oakville Master Servicing and Financing Scheme. Consequently, the front end financing of all projects at the Oakville Water Filtration Plant have been, and will be funded by the developers.

To ensure that the funds are used effectively, it is the purpose of this study to evaluate works which have been constructed under the current program of expansion to 109,000 cu. metre/day to accommodate an ultimate population of 200,000 all before moving to actual tendering of the expansion program. Since the timing of the construction of the works and the financing by developers is critical, it is essential that the program be set in a proper order of priority to ensure an orderly expansion program for the plant and the most effective use of the funds available.

Therefore, the purpose of this update study is to:

a) Consolidate the engineering and plant design aspect of the first phase expansion program that has been carried out since the issue of the first pre-design report.

b) Verify the water demand projections on the basis of historical water consumption records and forecast information on population commercial and industrial development, to be provided by the Region of Halton.

c) Evaluate and recommend any requirements for plant expansion in view of an evaluation of raw water quality data and the latest water demand projection.

d) Prepare functional designs, drawings and technical discussions and alternatives for work still to be designed and constructed as indicated on the chart of the expansion program to an ultimate population of 200,000.

e) Prepare functional cost estimates for the various alternatives and recommend works and confirm the staging of construction that has been identified in the above mentioned chart. In addition to this, the consultant is required to review the draft report - MASTER SERVICING AND FINANCING SCHEME FOR THE TOWN OF OAKVILLE prepared by Currie Coopers & Lybrand Limited and ensure that all projects have been identified and included in the master servicing report.

To enable you to prepare your submission, the following material has been enclosed herewith:

i) James F. MacLaren's Pre-Design Report, dated March 1977.

ii) James F. MacLaren's executive summary of the report - MODERNIZATION OF SUPERVISORY CONTROL, dated November 1980.

iii) The Oakville Water Treatment Plant Expansion chart, dated August 25, 1981

iv) Currie, Coopers & Lybrand draft report - MASTER SERVICING AND FINANCING SCHEME FOR THE TOWN OF OAKVILLE (to be returned to the Region).

v) The schedule of tasks to be performed.

The consultant should allow a minimum of five (5) meetings with the Regional staff.

SCHEDULE OF TASKS

1) Update the information in Chapter 1 of the Pre-Design report relating to raw water quality data, population and demand projections.

2) Summarize the existing plant design and capacity including works that have been constructed; now under construction and will be constructed in the future.

3) Undertake functional engineering evaluations of the following processes:

   a) Second intake expansion, as an alternative to sedimentation.

   b) Sedimentation consisting of contact tanks, and in future increasing the capacity by the addition of settling tubes.

   c) Sedimentation comprising the existing Accelator and new contact tanks (as an alternative to Item 3(b)).

   d) Reservoir storage capacity and the feasibility of constructing contact tanks on top of the future reservoir.

   e) Waste water treatment with and without the use of existing Accelator.

   f) Expansion of the existing high lift pumping station or provision of an additional new station.

4) Prepare hydraulic guidelines for the alternatives to be considered and the proposed works.

5) Update the plant layout drawing for the proposed works and assess the suitability of constructing contact sedimentation tanks, totally enclosed and housed with a superstructure on top of the future reservoir.

6) Review the supervisory and control system to be provided for the Oakville Water Treatment Plant and Booster Station as per recommendations of J.F. MacLaren report dated November 1980. Engineering of the proposed supervisory and control system is currently in progress, and being carried out by Simcoe Engineering Limited.

7) Document existing electrical facilities and identify future needs, including standby power requirement.

8) Prepare functional cost estimate for each alternative process and works to be recommended for this expansion program.

9) Based on the results of the aforementioned investigations, the existing charts on the phasing of the construction of the capital works would be reviewed and revised as necessary.

10) Prepare a comprehensive technical discussion and appropriate functional drawings on all matters investigated for submission to the Region. Ten (10) draft reports are to be submitted for review and comments and twenty-five (25) copies of the final report will be required.

REGIONAL MUNICIPALITY OF HALTON

SCHEDULE OF TASKS

FOR THE UPDATE ON STATUS OF OAKVILLE WATER TREATMENT PLANT IN PREPARATION FOR TENDERING OF WATER TREATMENT PLANT EXPANSION PROGRAM

ADDENDUM #1

3. (g) Review the raw and finished water quality and evaluate the merits of the existing chlorine dioxide treatment process for taste and odour problem. In addition, the consultant should review the current trend of the use of activated carbon (GAC and PAC) in water treatment plants around Lake Ontario. Also examine the economic viability of implementing such a treatment process at the plant, should this treatment process be required.

Appendix 2

Oakville Distribution Report Appendix 4

APPENDIX 4

TOWN OF OAKVILLE WATER DISTRIBUTION SYSTEM

HIGH LIFT PUMPING STATION

Oakville Water Filtration Plant

In order to assess the available supply and pressure from the existing high lift pumping station for the water distribution analysis of the Oakville Water Distribution System, it was decided to undertake pump tests at the Oakville High Lift Pumping Station. Information available at the commencement of the study indicated that capacity and total dynamic head of some of the units was in question. It was decided to undertake pump tests for each pump to determine the available head and flow.

On January 15, 1981, single pumping tests were done for Pumps No. 1, 3, and 4 and combined tests were done with Pumps No. 1, 3 and 4. Pump unit No. 2 was inoperable at the time of our testing and, as a result, a theoretical curve was used for analysis with respect to this pumping unit.

Table No. 1, attached hereto, gives a general summary of the nameplate information with respect to the four existing pumping units in the high lift station. Table No. 2 following indicates the rated points for each of the respective units. Initially each pumping unit was tested separately, the results of which are attached hereto on individual pump test tables for units 1, 3 and 4. Table No. 3 summarizes the actual available flow at various rated heads for the respective units considered.

With respect to each specific test, the following was concluded:

Pump No. 1

High Lift Pump No. 1 was installed in 1977 and is the newest unit. Pump tests were undertaken on this particular unit on April 22, 1978. These tests were undertaken at Babcock & Wilcox Canada Limited, Cambridge, Ontario. Results of these tests have indicated a slightly higher capacity from this particular unit than was actually indicated in the field. This may indicate that the flow results may be in question. From the certified shop tests taken, it would appear that the minimum total dynamic head tested was approximately 61.0 metres. The results of these tests indicate some decrease in capacity of this particular unit at higher flows.

Pump Noise

This unit was very noisy when the discharge valve was open. We expected this result since the discharge pressure with the valve fully open was 54 m (177 ft.). From the theoretical pump curve for this particular unit, it is indicated that this pump should not be operated at this low a head. It is obvious, assuming that the discharge valve is fully open and with this pump operating alone, that the rated point is considerably different than the actual operating point. It would appear that this particular unit should not operate below approximately 58 m (190 ft.) total dynamic head. It is recommended that this unit be operated with a throttled valve, or in parallel with at least one of the other units to maintain a suitable high discharge pressure. Other refinements have been considered that might be undertaken to resolve this particular problem. The only other feasible solution would be to decrease the size of the impeller. As discussed further in this report, it is important for future considerations to maintain this pump at as reasonable a high capacity as possible. It is recommended that this pump be operated at a minimum total dynamic head of 58 m (190 ft.), this being accomplished either by throttling with the discharge valve or by operating the pump only with other units.

Pump No. 2

Pump No. 2 could not be tested due to the fact that the pump was not in operational condition. The theoretical rated point for this pump, however, is considerably lower, 59.7 m (195 ft.), than the other units. The rated point is 70.4 m (231 ft.) for units 1 and 3. It is obvious that this particular unit is not suited to operate with the other three units in the pumping station, and, as a result, lends little support for supply. Because of the age and the small size of the unit, it is not recommended that any changes be made to this particular unit at this time.

Pump No. 3

Pump No. 3 was installed in 1961 and is nominally rated at 27,300 m^3/d (6 migd) at 70.4 m (231 ft.). The resultant tests performed on this particular unit indicate a test pumping curve considerably different from the theoretical curve. Results of this particular test indicated severe noise near the lower limits of operation. Discussions with operations' personnel indicated that this pump is normally operated against the completely open valve and operates under noisy conditions as experienced during the test. This could possibly have affected the theoretical capacity of this unit. Based on the pump tests performed by us on this date, it is recommended that this unit be rated as indicated in Table 4 as 22,500 m^3/d at 70.4 m TDH. This particular unit is used considerably as a low flow unit during the winter months; however, due to its age and apparent condition, we do not consider there to be any feasible or justifiable way of improving this particular unit.

Pump No. 4

The results of tests for Pump No. 4 as indicated in Figure No. 4 would suggest that the theoretical pump curve is considerably higher than the test results. This unit is rated at a lesser head than units 1 and 3 and, as a result, no apparent noise was experienced during the test. It would appear that a larger impeller in this particular unit would assist in the overall pumping capacity of the station; however, a change in impeller size would require a larger motor. As a result, it is not recommended that any changes be made to this particular unit. It is recommended that this unit be opened up and the impeller diameter and conditions checked.

Combined test curves were undertaken for Pumps 1, 3 and 4 running in parallel as indicated in Table 4.

Proposed Future Production Requirements
for the Oakville Water Purification Plant

It is proposed that the water purification plant be capable of supplying the maximum day rate to the distribution system. Balancing, fire protection and emergency storage will be supplied directly from Level 1, Level 2 and the proposed Level 3 storage facilities. In Table 5, attached hereto, we have projected the proposed requirements from 1980 to the ultimate system. We have indicated this using both a factor of 1.8 times the average day for maximum day and a factor of 2 times the average day for maximum day. Based on historical information, it is our belief that a factor of 1.8 would be satisfactory for the production requirements of the water purification plant.

Figure No. 1 indicates the various combinations of pumping units that were tested and the available head that can be expected. The following general conclusions can be reached from this analysis:

1. The available firm capacity of this station, assuming units 1 and 2 were out of service, would be approximately 65,000 m^3/d (13.3 migd). This is approximately equal to the present day maximum day demand. Assuming No. 2 pump were operational, the firm capacity of this station could be based on pumps No. 2, 3 and 4, with No. 1 being the largest unit out of service. Based on this approach, the present day firm capacity at the existing pumping station is approximately 82,500 m^3/d (18 migd). Based on the proposed projected requirements, a capacity of 18 migd would be satisfactory to about the year 1984 or 1985.

2. It should be stressed from the analysis performed that the existing distribution system and existing piping within the water purification plant high lift is capable of supplying at least a capacity of 107,000 m^3/d (23.5 mgd) at a rated TDH of 70 m (230 ft.) TDH.

In terms of the ultimate arrangement, it is recommended that six units be included for the high lift pumping station, four in the existing building and two additional units in a building addition. It is believed that an addition to the existing building could be easily made as indicated by the attached plan prepared some years ago. No investigation has been made of this point at this time.

Based on Table 5, a capacity of 103,000 m^3/d (22.7 mgd) would be satisfactory to supply the Oakville Water Distribution System up until the year 1991.

In order to achieve this capacity, two additional pumping units would have to be added to the existing high lift station in place of units 2 and 3. Some suction and discharge piping changes would be required in order for the station to operate satisfactory to this level of service; however, we believe that the amount of changes would be insignificant in relation to a similar cost for constructing a new high lift pumping station. Based on this approach, it is estimated that the maximum cost for replacing two of the existing units would be approximately \$250,000 per unit, for a total price of \$500,000. The proposed replacement of the units could be done in stages, as summarized in Table 6. At this time it is recommended that the existing system could be satisfactory to the year 1983; however, since the existing unit No. 2 is so incompatible with the remainder of the pumping units, it is recommended that this unit be replaced as quickly as possible with the 35,000 m^3/d unit. Once this unit is installed, the existing station would be adequate to the year 1987 or thereabouts. This would cost, in our estimation, approximately \$250,000.

An additional unit could be supplied in 1987, which we believe would satisfactorily meet the requirements of the system to the year 1991, at which time pumping station additions would be required where two new 35,000 m^3/d units could be installed.

Future Pump Selection

Care must be taken to ensure that future pumps are selected in such a manner that they will operate successfully over the necessary pumping ranges that can be expected at this station. We would recommend that future pumping units be selected, as follows:

1. That the rated point of a unit for maximum day consumption be selected as 70 metres.
2. That the evaluation for efficiency be based on an average day rate which would be somewhere in the order of 55 to 60 metres.
3. The operating range of the unit be such that it can operate over the full range to be expected, which in this case should be a minimum operating level of 50 metres and a maximum operating level of say 75 metres.

All three of these considerations must be evaluated when a pump is being selected. Typically, the unit will operate at a total dynamic head considerably less than the rated point. As a result, the maximum efficiency point should be so specified as to be that point at which the pump will normally be run. The rated point must be selected as that point for which the pump will have to overcome the greatest amount of head.

Recommendations and Conclusions

Based on this review of the Oakville High Lift Pumping Station with respect to the system requirements, it is recommended that the following points be considered:

1. Due to noise in pumps No. 1 and 3, these pumps should be operated either against a partially closed valve, or be operated in parallel with other units in the station. This will guarantee a better life for the units; however, occasionally running in the cavitation range would not unduly affect the units.

2. From the tests performed on January 12th, 1981, it has been indicated to us that the existing high lift pumping station with suitable equipment should be capable of supplying at least 104,000 m^3/d (23 migd) without major changes in piping, etc.

   As a result, it is recommended that two additional units be supplied in this station in order that this station can be rated at a firm capacity of 103,000 m^3/d (23 migd).

TABLE 1

OAKVILLE WATER SYSTEM
PUMPING STATIONS REQUIREMENTS

Name: HIGH LIFT PUMPING STATION

Location: Oakville Water Purification Plant

Zone Served: 1

	Unit No. 1	2	3	4
Pump Data	1977	1953	1961	1971
Manufacturer	Ingersoll-Rand	DeLaval	DeLaval	DeLaval
Serial No.		258352	700437	705080
Model No.	12HLV-2	12/10	L/12/10	P 18/14
Rated Head - m	70.4	59.7	70.7	64.0
Rated Flow m^3/d	36,000	26,500	27,300	42,300
Impeller Diameter (mm)				58.74
Motor Data				
Manufacturer	CGE	CGE	CGE	Bruce-Peebles
HP	500	250	350	500
RPM	1185	1750	1755	1180
Phase/Cycle/Volts	3/60/4160	3/60/4160	3/60/4160	3/60/4160

TABLE 2

OAKVILLE HIGH LIFT PUMPING STATION

THEORETICAL PUMP PERFORMANCE

PUMP NO.	RATED CAPACITY		RATED CAPACITY	
	m³/d	metres	migd	feet
1	36,000	70.4	7.9	231
2	26,500	59.7	5.8	195
3	27,300	70.4	6.0	231
4	42,300	64.0	9.3	210

TABLE 3

OAKVILLE HIGH LIFT PUMPING STATION

SUMMARY OF TEST RESULTS

PUMP NO.	1. m³/d/migd	2. m³/d/migd	3. m³/d/migd	4. m³/d/migd
Flow at 70.4 metres	34,000/7.5	7,000/1.5*	22,500/4.9	29,500/6.5
Flow at 64.0 metres	37,500/8.3	17,500/3.9*	23,500/5.2	36,000/7.9
Flow at 59.7 metres	39,500/8.7	22,000/4.8*	24,000/5.3	39,000/8.6

*Theoretical results.

TABLE 4

OAKVILLE HIGH LIFT PUMPING STATION

Summary of Pump Tests in Parallel

PUMPS OPERATING	DISCHARGE FLOW			TOTAL DYNAMIC HEAD		
	L/s	m^3/d	migd	m	feet	psi
3 & 4	740	63,900	14.1	55.7	183	79
1 & 3	750	64,800	14.3	54.5	179	77
1 & 4	900	77,800	17.1	60.5	199	86
1,2 & 3	1100	95,000	20.9	65.4	215	93
1,2,3, & 4*	1170	101,000	22.2	67.5	221	96

* Theoretical curve used for Unit #2

TABLE 5

TOWN OF OAKVILLE

PROJECTED WATER PLANT ON PRODUCTION REQUIREMENTS

	AVERAGE DAY		MAXIMUM DAY AT FACTOR OF 1.8 x AVERAGE		MAXIMUM DAY AT FACTOR OF 2.0 × AVERAGE	
YEAR	migd	m^3/d	migd	m^3/d	migd	m^3/d
1980	7.6	34,391	13.6	61,823	15.1	68,696
1986	10.6	48,105	19.0	86,476	21.1	96,093
1991	12.7	57,495	22.7	103,368	25.3	114,853
1996	15.0	68,219	27.0	122,658	30.0	136,287
2001	17.0	77,106	30.5	138,629	33.9	154,031
Ultimate	19.8	89,899	35.6	161,616	39.5	179,572

TABLE 6

RECOMMENDED RATING OF EXISTING UNITS

	EXISTING	REPLACED NO. 2 NEW UNIT	REPLACED NO. 3 WITH NEW m^3/d UNIT
1 at 70 m	34,000	34,000	34,000
2 at 70 m	7,000	35,000	35,000
3 at 70 m	22,500	22,500	35,000
4 at 70 m	29,500	29,500	29,500
Total at 70 m	93,000	121,000	133,500
Firm at 70 m	59,000	86,000	98,500
Firm at System Head	82,000	95,000	103,000
Year to which Pumps Adequate	1984	1987	1991

Simcoe Engineering Limited
Consulting Engineers
Pickering, Ontario

PUMP TEST TABLE

LOCATION: Oakville WTP
PUMP NO.: 1
MANUFACTURER: Ingersoll-Rand
SERIAL NO.:
YEAR OF MANUFACTURE: 1977

RATED FLOW: 416 L/s
RATED HEAD: 70.4 m
MOTOR SIZE: 500 HP
IMPELLER SIZE:
DATE OF TEST: 81/01/15
10:45 a.m.

TEST FLOW Units/ l/s	DISCHARGE PRESSURE Units/ m H_2O	SUCTION PRESSURE Units/	TOTAL DYNAMIC HEAD Units/ (m H_2O)	TOTAL DYNAMIC HEAD BASED ON PUMP CURVE (m H_2O)
Shut In	94.9		94.9	83.5
210	91.4		91.4	83.0
270	86.5		86.5	81.0
310	83.0		83.0	78.5
420	66.8		66.8	70.0
450	61.2		61.2	66.5
490	52.0		52.0	60.5

Comments: Suction pressure ≃0 psi
High lift magnetic flow meters used for flow measurement
(typical for all tests)

Simcoe Engineering Limited
Consulting Engineers
Pickering, Ontario

PUMP TEST TABLE

RPM: 1775
LOCATION: Oakville WTP
RATED FLOW: 316 l/s
PUMP NO.: 3
RATED HEAD: 70.4 m
MANUFACTURER: De Laval
MOTOR SIZE: 350 HP.
SERIAL NO.: C 12/10 700437
IMPELLER SIZE:
YEAR OF MANUFACTURE: 1961
DATE OF TEST: 81/01/15
10:20 a.m.

TEST FLOW Units/l/s	DISCHARGE PRESSURE Units/m H_2O	SUCTION PRESSURE Units/	TOTAL DYNAMIC HEAD Units/m H_2O	TOTAL DYNAMIC HEAD BASED ON PUMP CURVE/m H_2O
Shut In				
100	93.7		93.7	86.5
150	91.7		91.7	84.0
220	83.5		83.5	81.5
240	79.4		79.4	80.5
260	74.3		74.3	79.5
280	50.8		50.8	78.0

Comments: Discharge Pressure adjusted by 4.9 metres to reflect readings on other pressure gauges.

Simcoe Engineering Limited
Consulting Engineers
Pickering, Ontario

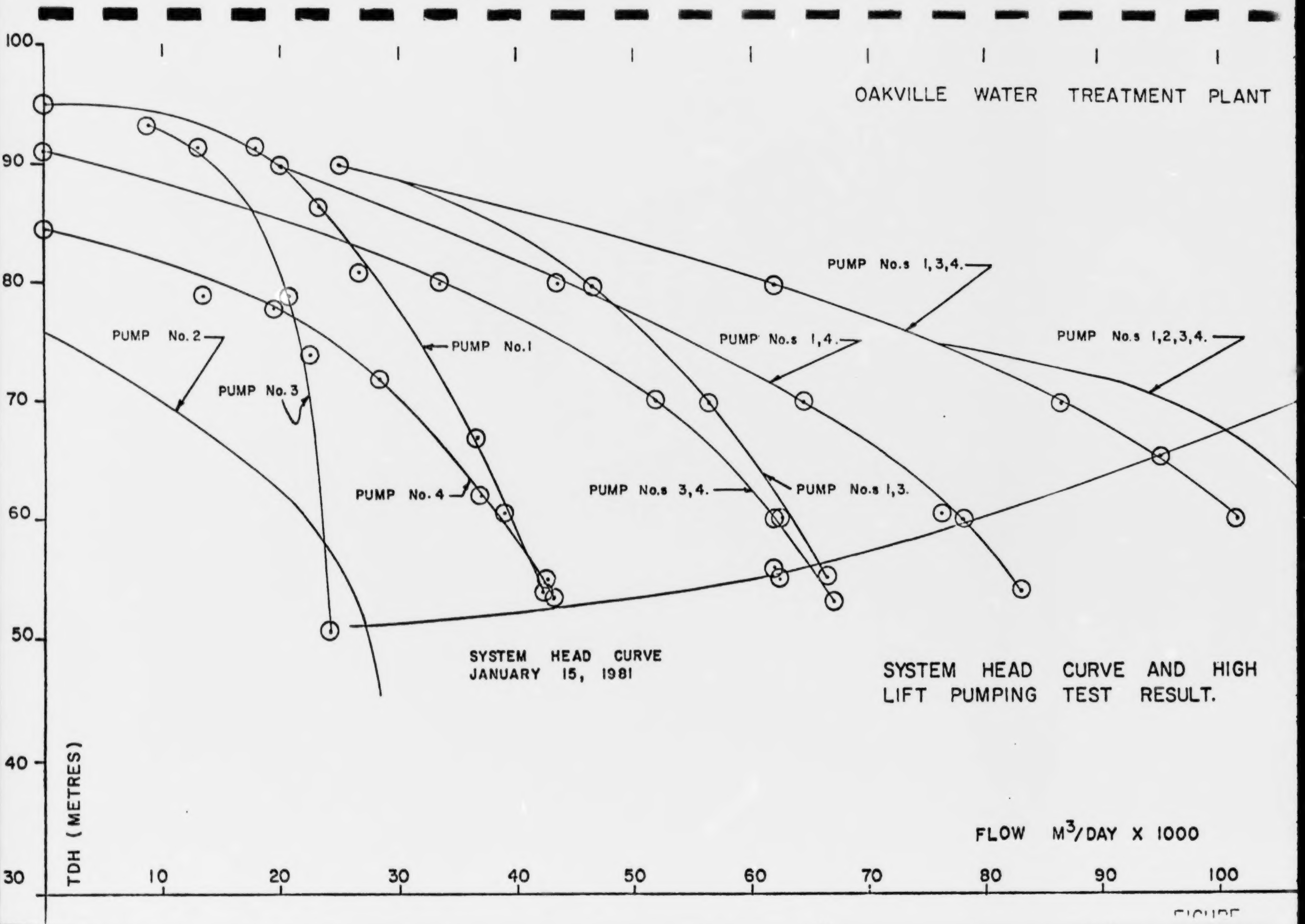
PUMP TEST TABLE

LOCATION: Oakville WTP
PUMP NO.: 4
MANUFACTURER: De Laval
SERIAL NO.: 705080 P 16/140
YEAR OF MANUFACTURE: 1971

RPM: 1185
RATED FLOW:
RATED HEAD: 64 m
MOTOR SIZE: 500 HP
IMPELLER SIZE:
DATE OF TEST: 81/01/15
11:05 a.m.

TEST FLOW Units/ l/s	DISCHARGE PRESSURE Units/m H_2O	SUCTION PRESSURE Units/	TOTAL DYNAMIC HEAD Units/(m H_2O)	TOTAL DYNAMIC HEAD BASED ON PUMP CURVE (m H_2O)
Shut In	84.4		84.4	90
160	79.5		79.5	87
230	78.0		78.0	85
330	71.7		71.7	81
430	61.9		61.9	74
480	54.8		54.8	70
500	52.7		52.7	69

Comments: Suction pressure ≈0 psi
High lift magnetic flow meters used for flow measurement
(typical for all tests)



OAKVILLE WATER TREATMENT PLANT

SYSTEM HEAD CURVE AND HIGH LIFT PUMPING TEST RESULT.

Appendix 3

Oakville Hydro Correspondance

Simcoe

82 02 19
RR 108.43

Oakville Hydro
2350 Trafalgar Road
P.O. Box 1900
Oakville, Ontario
L6J 5E3

Attention: Mr. A. L. Johnson, C.E.T.

Dear Sirs:

Re: Oakville Water Plant Electrical Supply

Further to our recent meeting of February 16th, 1982, re: Oakville Water Purification Plant, we require clarification of the following points:

1. The capacity available to the water plant from the 5 kV supply from Margaret Street, including any restrictions of its use.
2. The method and procedures required to switch the two incoming lines, Margaret Street and Kerr Street.
3. The capacity available to the water plant from the 27.6 kV supply from Kerr Street.
4. If, in future, we require an additional transformer to be located next to the present 1500 kVA transformer, owned by PUC, will this be supplied and installed by the Oakville PUC.

We would also like to draw your attention to the fact that the present set-up allows the water plant to receive power via the 5 kV Margaret Street line which by-passes the PUC metering. We realize that as this set-up was installed during the time that the PUC operated the facility, there was no need for billing. As this situation no longer exists, perhaps you will require changes in the system.

We would be obliged if you could advise us as to what action will be taken by you or what action we will be required to take to remedy this situation.

Simcoe Engineering Limited ◦ Consulting Engineers
Simcoe Building, 345 Kingston Rd, Pickering, Ontario L1V 1A1 ◦ (416) 286 - 2285

If you have any questions, or require further information, please do not hesitate to contact us.

Yours very truly,

SIMCOE ENGINEERING LIMITED

T. Connor
Electrical Department

TC/jr
cc: Mr. C. F. Kong, P. Eng.
Mr. A. Smith
Region of Halton

OAKVILLE HYDRO

P.O. Box 1900, 2350 Trafalgar Road
Oakville, Ontario L6J 5E3 Phone 845-3461

1982 04 06

Simcoe Engineering Limited
345 Kingston Road
PICKERING, Ontario
L1V 1A1

Attention: Mr. T. Connor
Electrical Department

Dear Sirs:

Subject: Oakville Water Plant
Electrical Supply
Our file: 157.6
Your file: RR 108.41

Your letter of February 19, 1982 to Mr. A. L. Johnson, has been forwarded to me for a reply.

The peak load for the above customer was 1536 kVA during April 1981.

The normal supply to the water plant is from our substation on Kerr Street. The alternative supply is from our Margaret M.S. During peak load conditions, the feeder from Margaret M.S. has a reserve capacity of approximately 950 kVA. This makes the transfer of load during peak conditions, impossible.

Due to the present arrangement of the Region's 5 kV switchgear, power from only one source can be used at any one time. Transfer to the alternative supply can only be made at off-peak times and must be approved in advance by our Engineering and Operations Manager or his delegate. Our Customer Service Manager must be notified of the load transfer duration and amount, since there is no metering on the Margaret M.S. supply.

Our present policy is to supply distribution voltage to customers up to a level of 500 kVA. After that point, it is necessary for the customer to supply his own substation, taking power from our 27.6 kV system.

...2/

Since this customer is presently supplied from a municipal substation which is not fully loaded, we are prepared to continue to supply power to it at 4160 V until the load reaches the capacity of a municipal feeder. This we consider to be 2000 kVA. When this load is reached, it will be necessary for the Region to install a 27.6 kV substation to supply the water plant load.

Our Capital Works forecast indicates that we will be reconstructing our municipal substation on Kerr Street in 1986. At this time, we will be extending our underground system south from Lakeshore Road. By then we anticipate that the full capacity of our stations will be required to supply the needs of our customers in the Kerr Street and Lakeshore Road area and the Region should have its own station in place.

Our 27.6 kV system on Kerr Street is capable of supplying up to 5000 kVA to a customer-owned station.

Existing service from Margaret M.S. should be considered as only an emergency supply and at reduced capacity during peak times.

If you require further information, please contact us.

Yours truly,

R. M. Schwandt

R. M. Schwandt, P. Eng.
Chief Design Engineer

pc: Region of Halton
Att: D. R. MOrrier, P. Eng.

RMS/lm

Simcoe

82 06 11
RR 108.41

Oakville Hydro
2350 Trafalgar Road
P.O. Box 1900
Oakville, Ontario
L6J 5E3

Attention: Mr. R. M. Schwandt, P. Eng.
Chief Design Engineer

Dear Sirs:

re: Oakville Water Plant
Electrical Supply
Your file: 157.6
Our file: RR 108.41

Further to our meeting of April 28, 1982, I would like to clarify/confirm the following points:

1. As stated in your letter dated 1982-04-06, you will be constructing a new substation on Kerr Street in 1986. Therefore, the Region will have to construct a new substation, within the Water Plant as the existing Municipal substation on Kerr Street will no longer be available for the Water Plant use.

2. Also, as stated in your letter dated 1982-04-06, you will be extending your underground service south from Lakeshore Road to the new Kerr Street Municipal substation. Therefore, my understanding of the proposed systems is, by 1986, there will be available (for Water Plant use) two separate 27.6 KV supplies at the Kerr Street Municipal substation.

With the above two points in mind, we would propose that the system shown and outlined in our enclosed sketch Figure No. 7 could be used to satisfy both of our requirements.

As discussed at our recent meeting, we have assumed that you will provide us with a connection manhole on the west side of the new Kerr Street substation. We will be responsible for all cabling from this manhole to the new in-plant substation.

Also as discussed at our recent meeting, we have based our "reliability calculations" on a maximum of 1-hour to switch incoming lines in the event of a line failure. We would be obliged if you could provide us with more specific data on this point.

If you have any comments on any of the above points, or if there are any points, financial or technical that we may have missed, please do not hesitate to contact us. We thank you for your co-operation and assistance.

Yours very truly,

SIMCOE ENGINEERING LIMITED

T. Connor

TC:mc

c.c. C. F. Kong, P. Eng.
Region of Halton

Appendix 4

Electrical System Reliability

APPENDIX 4

OAKVILLE WATER PURIFICATION PLANT

ELECTRICAL SYSTEM RELIABILITY ANALYSIS

INTRODUCTION: This appendix provide details of the reliability analysis conducted for the power supply and distribution system proposed for the Oakville Water Treatment Plant, as described in Section 11 of this report. The anaylsis was conducted to assess the need for standby power at the plant.

STUDY METHOD: This analysis is based on figures derived from extensive surveys carried out by the Institute of Electrical & Electronics Engineers (IEEE) completed in 1972, which included a total of 1982 equipment failures reported by 30 companies covering 68 plants in 9 industries in the U.S.A. and Canada. In 1980 the survey results were brought together in a new IEEE Standard #493-1980, titled "IEEE Recommended Practice for the Design of Reliable Industrial and Commercial Power Systems", together with detailed suggestions for applying the information for evaluating the reliability of new or existing electrical systems.

The method of reliability evaluation used is referred to as the "minimal cut-set method", which is fully described in the above Standard. In essence this method recognizes that when any number of system components are "in series" with one another, each and every one must operate or the system will fail; but on the other hand if two components (or two strings of series components) are "in parallel" then as long as either component (or series string) is operational the system can continue to function, though not necessarily at full capacity. The results of the study are presented in Tables 1 and 2.

EXPLANATION OF TABLES: Tables 1 and 2 list the essential components in the proposed system between the Hydro supply and the pump drive motors (4160 V and 600 V respectively). Opposite each component is shown the statistical failure rate "λ" (failures per unit-year) and the number of hours of downtime per failure "r". The product of these two figures λ.r then represents the total number of hours of downtime per year predicted for that component. The sum of all of the r figures for a series string of components indicates the total number of interruptions per year due to failures in that string; and similarly the sum of all of the "λ.r" figures indicates the total number of downtime hours per year predicted.

The two (ultimately three) 2000 kVA, 27.6 kV/4160 V transformers together with their switching components and tie switch provide redundancy and could be considered to operate in parallel if it were not for the normally-open tie switch. However, it is reasonable to assume that in the event of a failure of any component in one transformer circuit, it could be isolated, and the tie switch closed to provide power from the remaining transformer within one hour. Thus instead of using the sum of the λ.r figures to arrive at system downtime, a figure of 1 hr. is used for r and multiplied by the sum of the λ figures to obtain a virtual λ.r figure (shown in brackets for item 10 Table 1). This indicates that although the failed transformer circuit may be out of service (for even 1076 hrs. as indicated in item 7, Table 1) the system would be back in service in 1 hour. The same argument applies to the redundant circuits through the two 500 kVA 4160/600 V transformers (see item

6, Table 2).

INTERPRETATION: The bottom line in Table 1 indicates that, on the average, any 4160 V drive motor will be out of service about twice (2.0612 times) per year as a result of a failure in the electrical system, or the motor itself; and the total downtime in a year will be 5.67 hours. By far the most frequent cause of downtime will be outages on the Hydro 27.6 kV primary feeders line (1.644 times per year). With the proposed double-circuit feeeder it is expected that the manual throw-over to the alternate line can be accomplished within one hour, and so the downtime per failure is shown as one hour.

The probability of losing both 27.6 kV feeders at the same time, according to the IEEE surveys, is 0.312 times /yr (or about once every three years) but the average time to restore service is only 0.52 hours.

It will be noted that the number of downtime hours per failure of some items is quite large-for example, 17.3 hours for a failure in the 27.6 kV bus; but since the number of failures per year is only 0.0019 (which translates to one failure in about 526 years) the risk is not considered to be significant.

In the case of the 27.6 kV/4160 V transformer, even though the failure rate is low (0.0130 or about once in 77 years) the downtime of 1076 hrs would be unacceptable if an alternate were not available, and this is the reason for the proposed arrangment with two transformers and tie switch. It is assumed that a faulty transformer could be isolated and 4160 V power restored via the tie switch within one hour. The probability of both transformer legs (items 5 through 10) failing at the same time is calculated to be .0000998 times /yr, (or once in about 10 000 years), and is therefore ignored.

Table 2 shows the component failure rates and downtime figures through the system to the 600 V motors. It is interesting to note here that, in spite of the additional chain of components (transformer switch, circuit breaker etc.) necessary between the 4160 V bus and the 600 V bus, the failure rate and downtime figures for the average 600 V motor are very nearly the same as for the 4160 V motors. The reason for this is the considerably higher failure rate of the 4160 V motor itself, compared to that of the 600 V motor.

The reliability of the 600 V system is greatly enhanced by having two 500 kVA 4160/600 V transformers with a tie switch for interconnecting the two 600 V buses in the event of a transformer failure.

GENERAL COMMENT: It shall be noted that all of the figures presented are based upon averages from all industries surveyed. In most cases the figures from the surveys differed very significantly from one industry to another. For example the all-industry average failure rate of high-voltage motor starters was 0.0153 per unit-year. For the chemical industry it was only 0.0026, but for the petroleum industry 0.1294. Because of such spreads, and because water treatment and supply facilities were not singled out as a separate industry in the surveys, the all-industry average figures were considered to provide the most reliable estimates of probable failure rates and durations.

CONCLUSION: Based on the analytical results, it is clear that the proposed power system is very highly reliable, and the provision of any standby power system for plant operations is unnecessary.

OAKVILLE WATER TREATMENT PLANT

RELIABILITY INDICES FOR PROPOSED ELECTRICAL SYSTEM
(To 4160 V Motor Shaft)

COMPONENT	FAILURES PER YEAR	HRS. DOWN-TIME PER FAILURE**	FORCED HRS. DOWNTIME PER YEAR
1. 27.6 kV Feeder from Oakville Hydro, Double Circuit with manual throw-over (loss of 1 source only)	1.644	1.00*	1.6440
2. Loss of both 27.6 kV sources simultaneously	0.312	0.52	0.1622
3. 27.6 kV Disconnect Switches, enclosed (2)	0.0122	3.60	0.0439
4. 27.6 kV Switchgear Bus (connected to one instrument trans. compartment and two fused switch compartments) Total to line side of transformer primary disconnect switch	0.0019		0.0329
5. 27.6 kV Fused Disconnect Switch, enclosed	0.0061	3.6	
6. Fuses for item (5)	0.0019	5.5	
7. Transformer 27.6:4.16 kV, 2500 kVA	0.0130	1076.0	
8. 4160 V Circuit Breaker, or fused switch, metalclad	0.0036	109.0	
9. 4160 V Cable in Conduit, 50 ft.	0.0025	140.0	
10. 4160 V Cable terminations (2)	0.0018	198.0	
(Sub-total for tramsformer leg (item 5 through 10) assuming switchover to alternate leg can be completed in one hour)	(0.0289)	(1.0)	(0.0289)
Total to 4160 V-bus	1.9990		1.9119
11. 4160 V Bus (connected to 3 starters & 2 switches)	0.0032	17.3***	0.0554
12. 4160 V Motor Starter	0.0153	23.6	0.3611
13. 4160 V Cable 30'(typ) in conduit	0.0015	140.0	0.2100
14. 4160 V Cable Terminations (2)	0.0018	34.6	0.0623
15. 4160 V. Induction Motor	0.0404	76.0	3.0704
Total to Shaft of 4160 V Motor	2.0612		5.6711

NOTES:

* Assuming average time to transfer to alternate source takes 1 hr.

** Industry average figure, unless otherwise noted.

*** Repair failed component.

TABLE 2

OAKVILLE WATER TREATMENT PLANT

RELIABILITY INDICES FOR PROPOSED ELECTRICAL SYSTEM
(To 600 V Motor Shaft)

COMPONENT	FAILURES PER YEAR	HRS. DOWN-TIME PER FAILURE**	FORCED HRS. DOWNTIME PER YEAR
1. Total to 4160 V Bus (from TABLE 1)	1.9990		1.9119
2. 4160 V Bus (connected to 3 starters & 2 switches)	0.0032	17.3	
3. Transformer primary fused Disconnect Switch, 4160 V	0.0061	3.6	
4. Fuses for item (3)	0.0019	5.5	
5. Transformer 4160:600 V, 500 kVA	0.0037	61.0	
6. Transformer see Circuit Breaker, metalclad (Sub-total for transformer leg (items 2 through 6) assuming switchover to alternate 4160:600 V transformer leg can be completed in one hour)	(0.0179)	(1.0)*	(0.0179)
Total to 600 V Bus	2.0169		1.9298
7. 600 V Bus (connected to 6 circuit breakers)	0.0020	550.0	1.1000
8. 600 V Motor Starter	0.0139	65.1	0.9049
9. 600 V Cable 30' (typ)	0.00005	20.8***	0.0010
10. 600 V Cable Terminations (2)	0.00025	8.0***	0.0020
11. 500 V Induction Motor	0.0109	114.0	1.2426
Total to Shaft of 600 V Motor	2.0440		5.1803

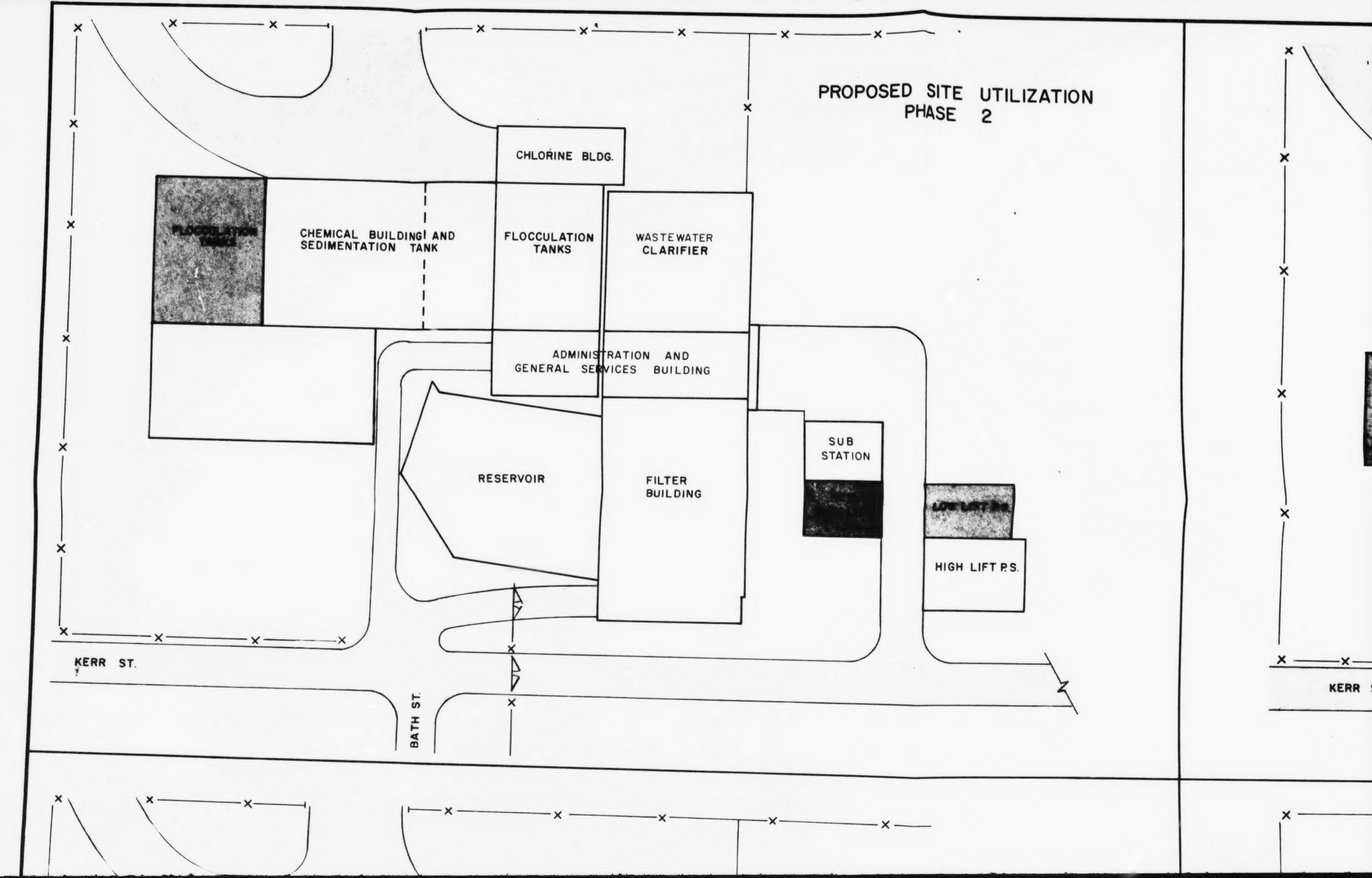
NOTES:

* Assuming average time to transfer to alternate source takes 1 hr.

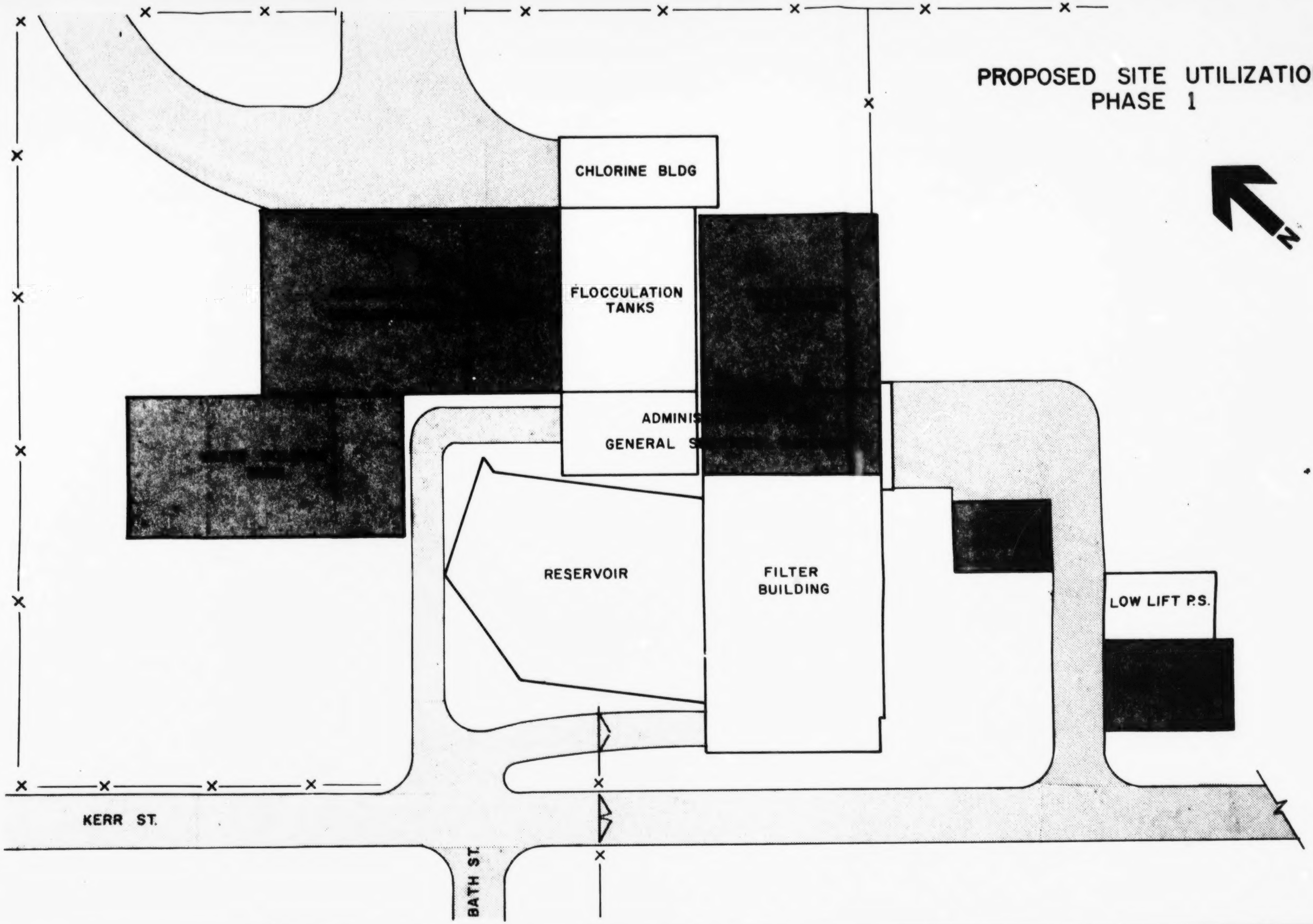
** Industry average figure, unless otherwise noted.

*** Repair failed component.

PROPOSED SITE UTILIZATION PHASE 2


CHLORINE BLDG.
FLOCCULATION TANKS
CHEMICAL BUILDING AND SEDIMENTATION TANK
FLOCCULATION TANKS
WASTEWATER CLARIFIER
ADMINISTRATION AND GENERAL SERVICES BUILDING
RESERVOIR
FILTER BUILDING
SUB STATION
LOW LIFT P.S.
HIGH LIFT P.S.
KERR ST.
BATH ST.

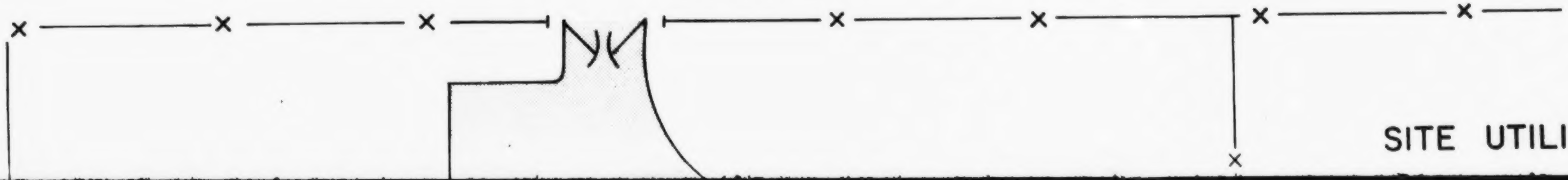
ZATION



PROPOSED SITE UTILIZATION
PHASE 1

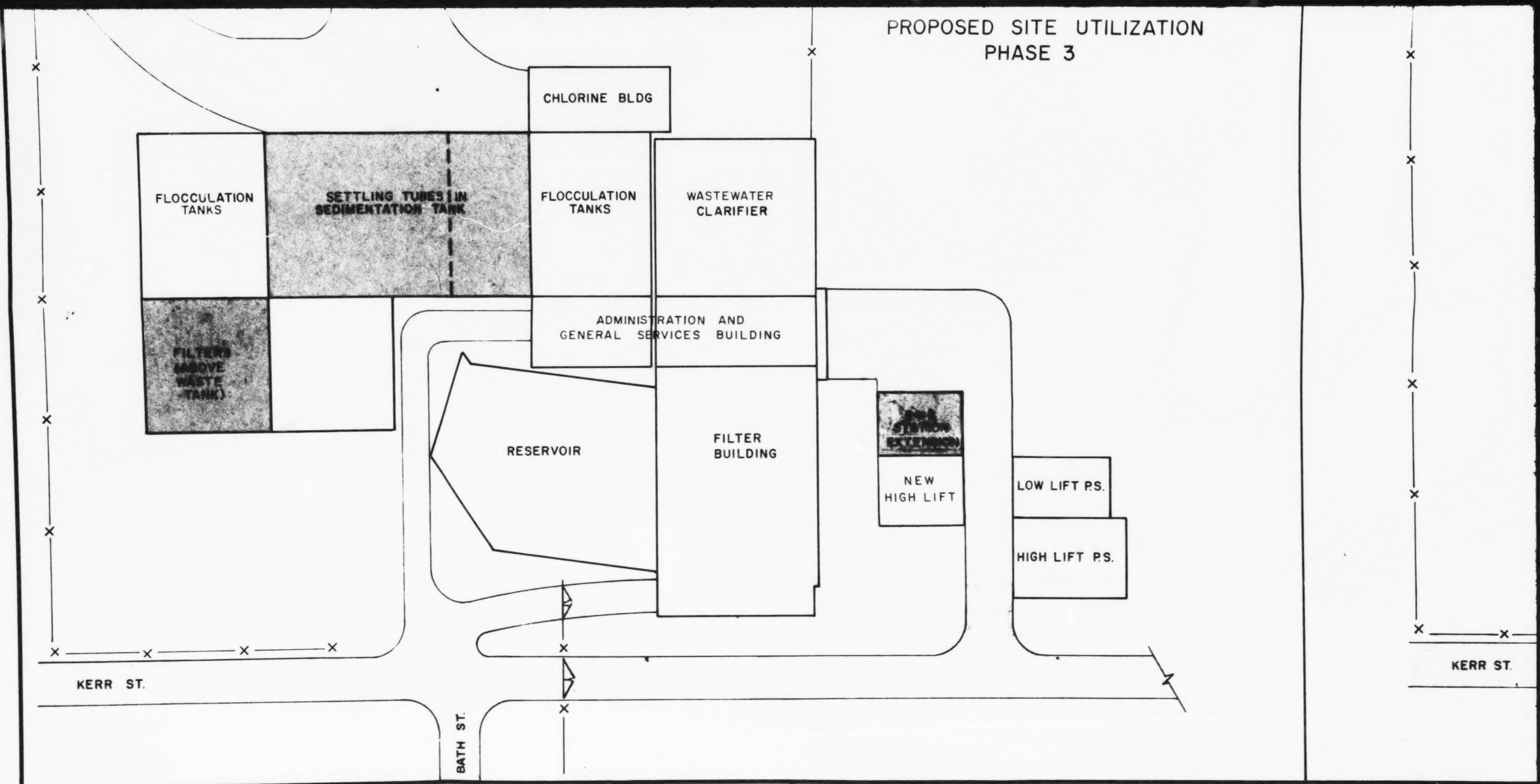
N
CHLORINE BLDG
FLOCCULATION TANKS
ADMINIS
GENERAL S
RESERVOIR
FILTER BUILDING
LOW LIFT P.S.
KERR ST.
BATH ST.

ZATION



SITE UTILIZATION - 1982

PROPOSED SITE UTILIZATION
PHASE 3


CHLORINE BLDG
FLOCCULATION TANKS
SETTLING TUBES IN SEDIMENTATION TANK
FLOCCULATION TANKS
WASTEWATER CLARIFIER
FILTERS (ABOVE WASTE TANK)
ADMINISTRATION AND GENERAL SERVICES BUILDING
RESERVOIR
FILTER BUILDING
STATION EXTENSION
NEW HIGH LIFT
LOW LIFT P.S.
HIGH LIFT P.S.
KERR ST.
BATH ST.
KERR ST.

No.	Revisions	By	Date

Drawn By: R. Mac LEA

Checked By: A. BOWERS

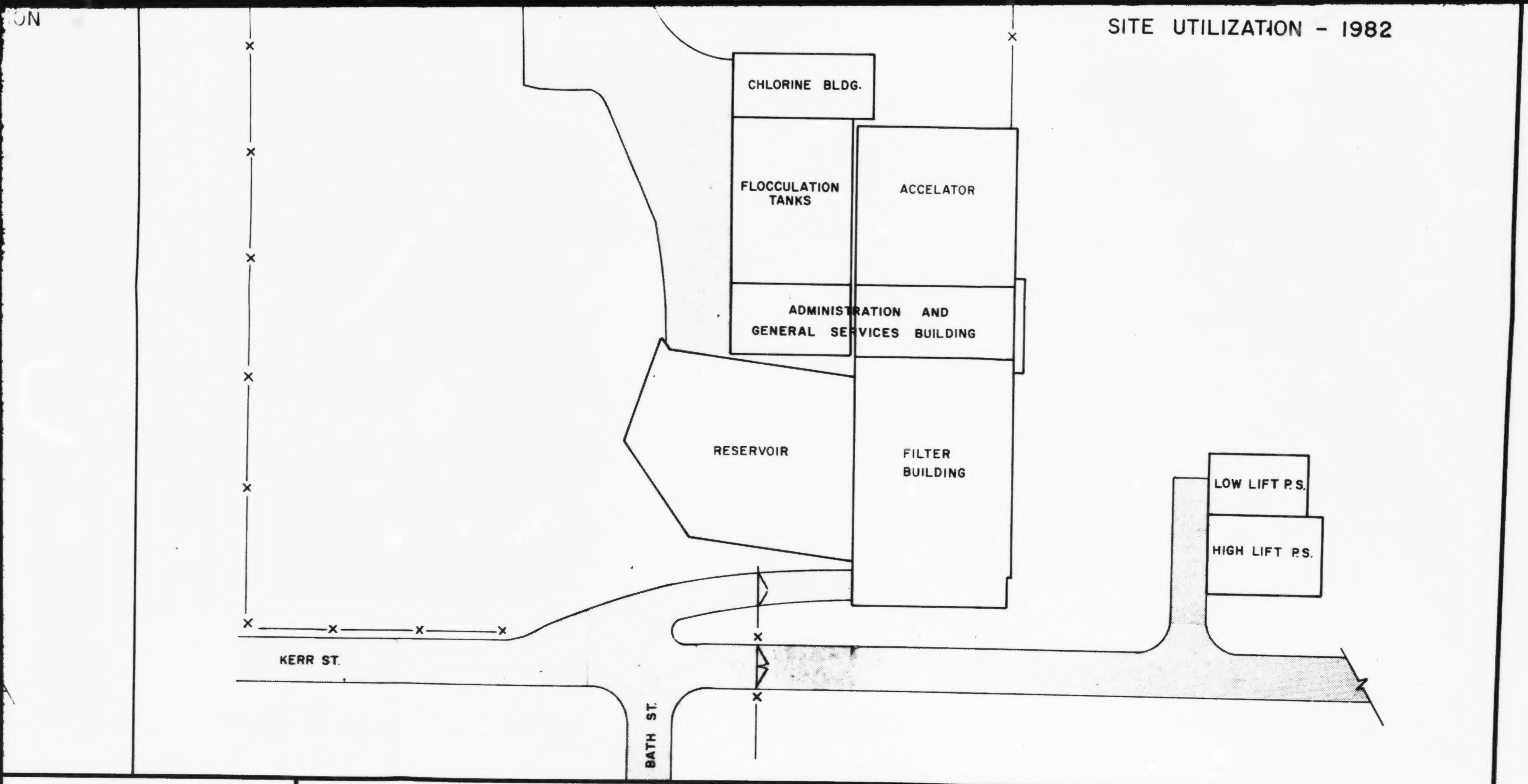
Date: APRIL 1982

Simcoe

Simcoe Engineering Limited • Consulting Engineers

Th

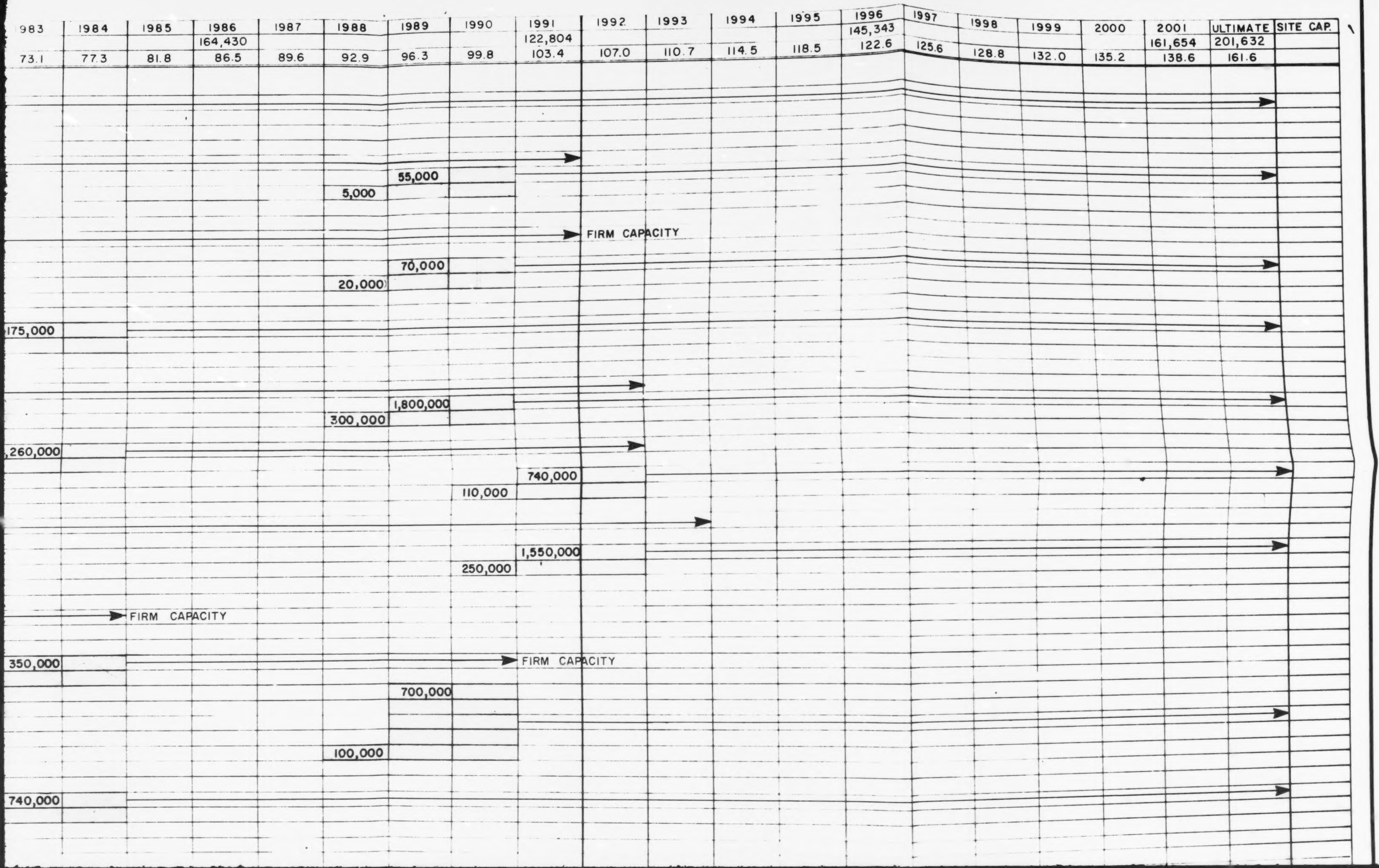
SITE UTILIZATION - 1982


ON
CHLORINE BLDG.
FLOCCULATION TANKS
ACCELATOR
ADMINISTRATION AND GENERAL SERVICES BUILDING
RESERVOIR
FILTER BUILDING
LOW LIFT P.S.
HIGH LIFT P.S.
KERR ST.
BATH ST.

ted • Consulting Engineers	The Regional Municipality of Halton DEPARTMENT OF PUBLIC WORKS OAKVILLE WATER PURIFICATION PLANT	Project Title: : OAKVILLE W.P.P. UPDATE REPORT Drawing Title: SITE DEVELOPMENT: 1982 - ULTIMATE

All Dimensions Are In Millimetres All Elevations Are Geodetic Metric	Scale: 1: 500	Drawing No. R1

ITEM Nº	PROCESS	UNIT	PR. Nº	WORK TO REACH CAPACITY	CAPACITY m3/d	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
	YEAR					1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
	POPULATION									72,439						164,430	
	MAXIMUM DAY CONSUMPTION 1000 m³/D									61.8	65.4	69.1	73.1	77.3	81.8	86.5	89.6
1	INTAKE	762 mm DIA.		EXISTING	300,000												
		1828 mm DIA		EXISTING													
2	LOW LIFT PUMPING STATION	SCREEN		EXISTING	109,000												
		SCREEN		NEW	109,000												
				ENGINEERING													
		PUMPS		EX. No. 1	54,500												
				EX. No. 2	27,250												
				EX. No. 3	54,500												
				EX. No. 4	27,250												
				NEW No. 5	54,500												
				ENGINEERING													
3	RAW WATER SUPPLY	LOW LIFT HEADER		DUPLICATE PART HEADER	ULTIMATE								175,000				
				ENGINEERING								25,000					
4	PRE-TREATMENT	FLOCCULATION TANKS		EXISTING 1 + 2	109,000												
				NEW 3 + 4													
				ENGINEERING													
5	PRE-TREATMENT	SEDIMENTATION TANK		NEW TANK	ULTIMATE								1,260,000				
				ENGINEERING								190,000					
				SETTLING TUBES	—												
				ENGINEERING													
6	FILTRATION	FILTERS		EX. No.'s 1-8	117,000												
				NEW 9-10													
				ENGINEERING													
7	HIGH LIFT	PUMPS		EX. No. 1	34,000												
				EX. No. 2	7,000										FIRM CAPACITY		
				EX. No. 3	22,500												
				EX. No. 4	29,500												
				REPLACE No.s 2 + 3	(70,000)								350,000				
				ENGINEERING								50,000					
				NEW STATION													
				NEW No. 5													
				NEW No. 6													
				SURGE TANK	ULTIMATE												
				ENGINEERING													
8	WASTEWATER TREATMENT	SYSTEM		HOLDING TANK/PIPING/MODS. TO ACCELATOR	ULTIMATE								740,000				
				ENGINEERING								110,000					


983
1984
1985
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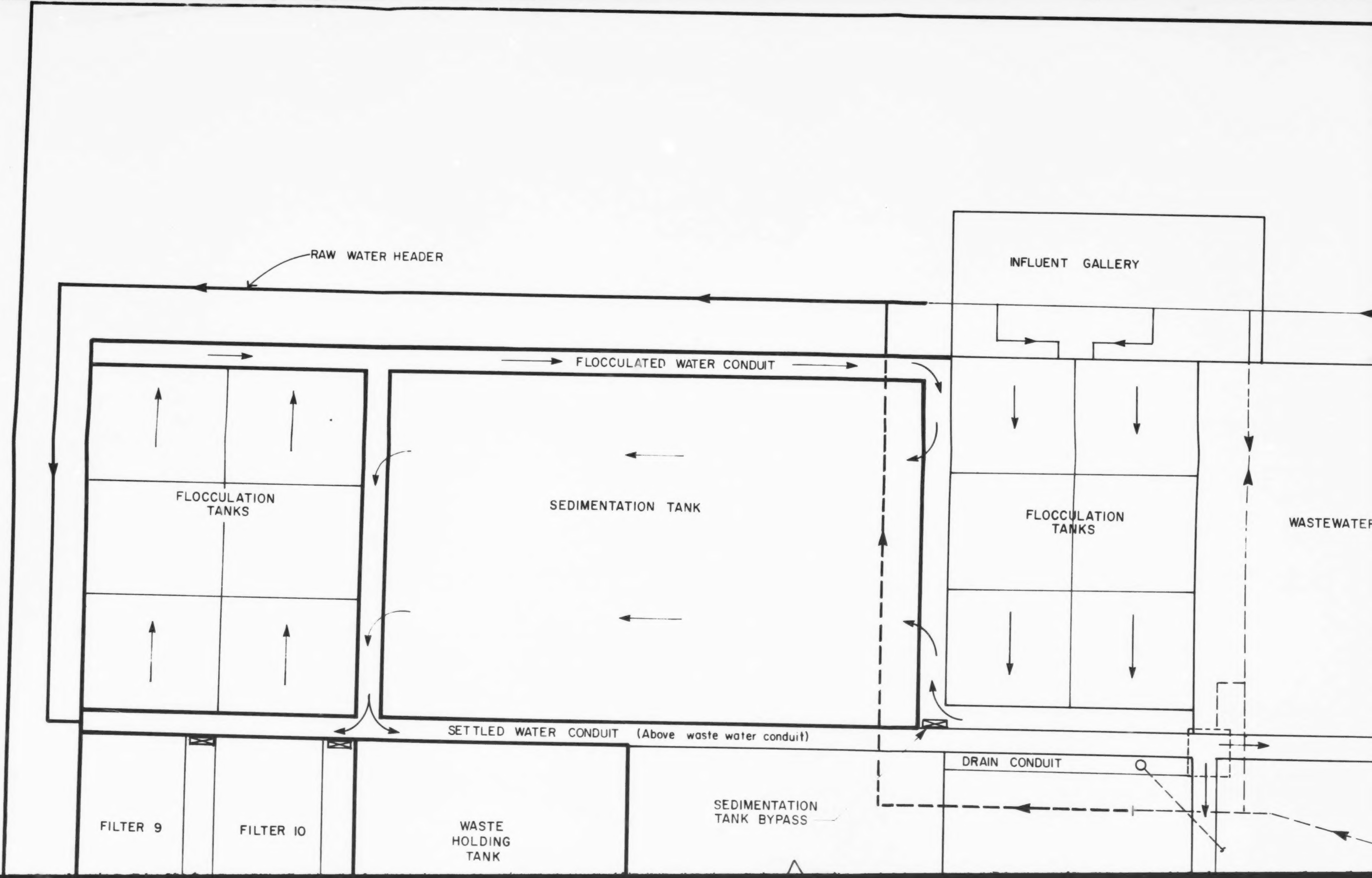
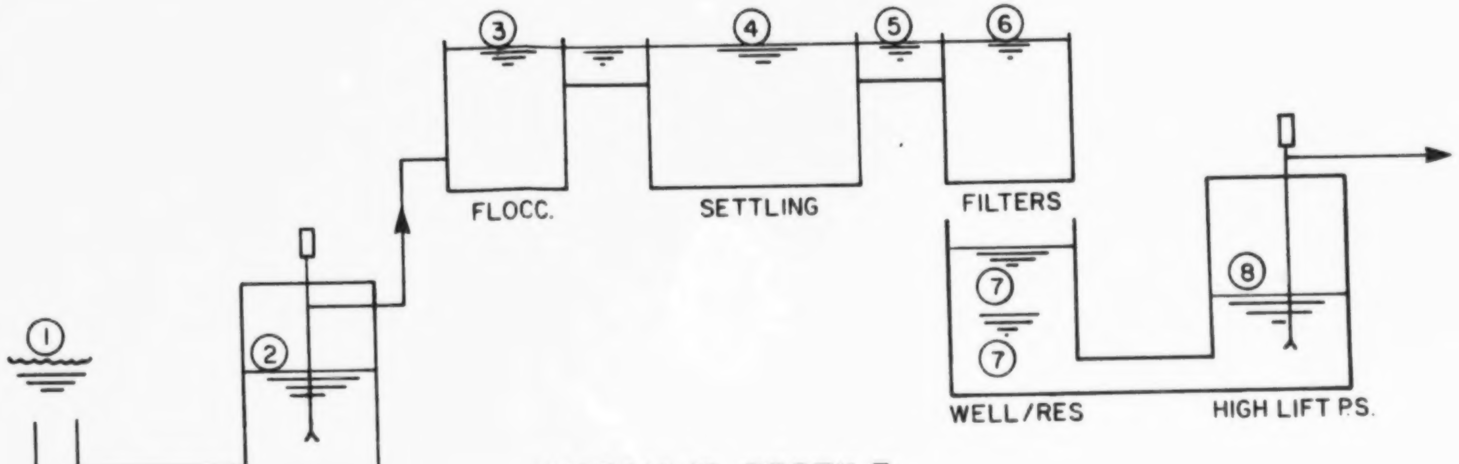
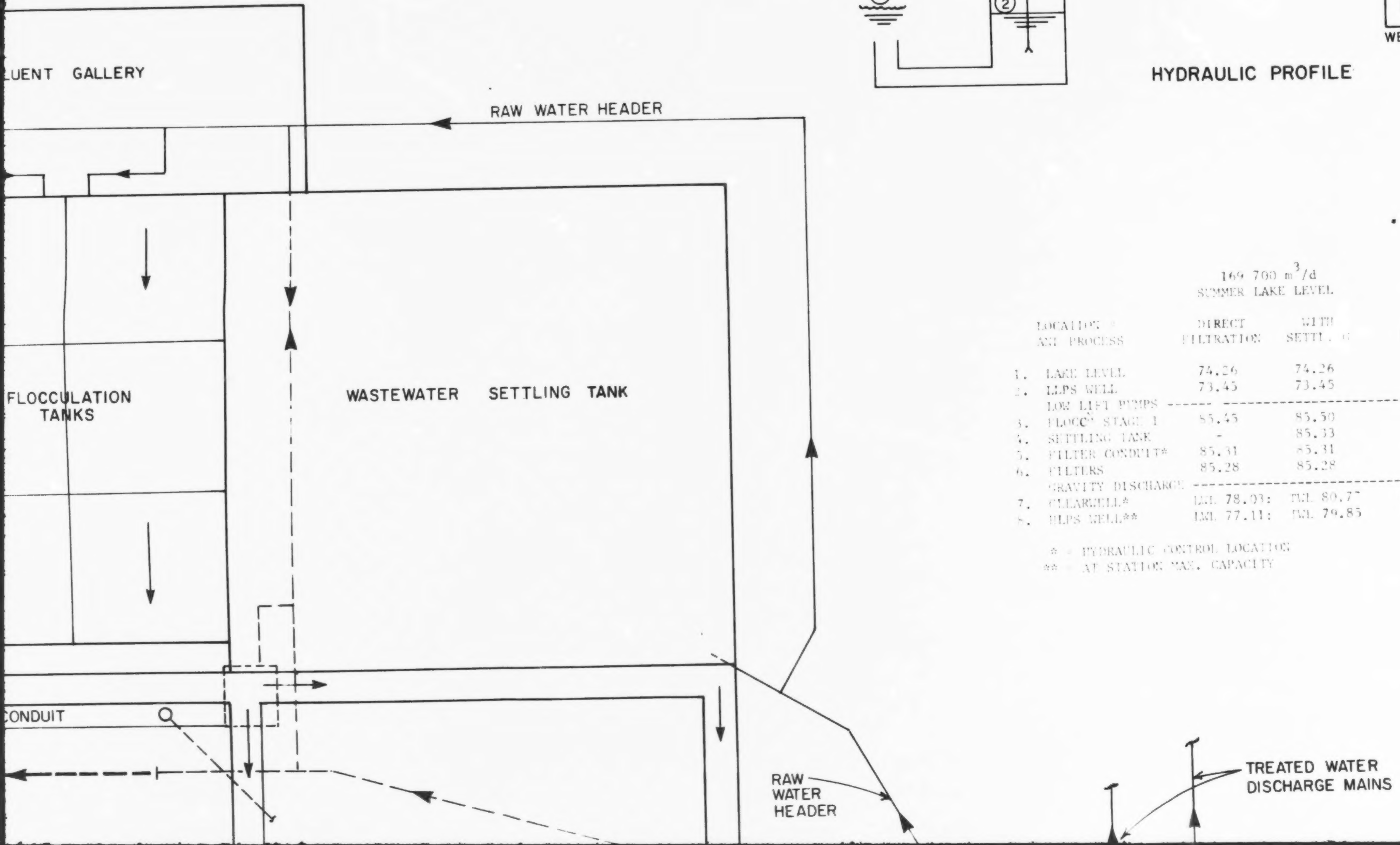
2000
2001
ULTIMATE
SITE CAP.
164,430
122,804
145,343
161,654
201,632
73.1
77.3
81.8
86.5
89.6
92.9
96.3
99.8
103.4
107.0
110.7
114.5
118.5
122.6
125.6
128.8
132.0
135.2
138.6
161.6
55,000
5,000
FIRM CAPACITY
70,000
20,000
175,000
1,800,000
300,000
,260,000
740,000
110,000
1,550,000
250,000
FIRM CAPACITY
350,000
FIRM CAPACITY
700,000
100,000
740,000


RAW WATER HEADER
INFLUENT GALLERY
FLOCCULATED WATER CONDUIT
FLOCCULATION TANKS
SEDIMENTATION TANK
FLOCCULATION TANKS
WASTEWATER
SETTLED WATER CONDUIT (Above waste water conduit)
DRAIN CONDUIT
FILTER 9
FILTER 10
WASTE HOLDING TANK
SEDIMENTATION TANK BYPASS


③
④
⑤
⑥
FLOCC.
SETTLING
FILTERS
①
②
⑦
⑦
⑧
WELL/RES
HIGH LIFT P.S.

HYDRAULIC PROFILE


LUENT GALLERY
RAW WATER HEADER
FLOCCULATION TANKS
WASTEWATER SETTLING TANK
CONDUIT
RAW WATER HEADER
TREATED WATER DISCHARGE MAINS

LOCATION & AND PROCESS	169 700 m³/d SUMMER LAKE LEVEL		94 400 m³/d AVERAGE LAKE LEVEL	
	DIRECT FILTRATION	WITH SETTLING	DIRECT FILTRATION	WITH SETTLING
1. LAKE LEVEL	74.26	74.26	73.62	73.62
2. LLPS WELL	73.45	73.45	73.37	73.37
LOW LIFT PUMPS				
3. FLOCC. STAGE 1	85.45	85.50	85.42	85.43
4. SETTLING TANK	-	85.33	-	85.38
5. FILTER CONDUIT*	85.31	85.31	85.37	85.37
6. FILTERS	85.28	85.28	85.36	85.36
GRAVITY DISCHARGE				
7. CLEARWELL*	LWL 78.03:	TWL 80.77	LWL 78.03:	TWL 80.77
8. HLPS WELL**	LWL 77.11:	TWL 79.85	LWL 77.64:	TWL 80.38

* = HYDRAULIC CONTROL LOCATION
** = AT STATION MAX. CAPACITY

	TREATMENT		TO [illegible]								110,000						
			ENGINEERING														
9	POWER SUPPLY	SUB STATION	EXISTING												REMOVED FROM SER		
			NEW SUBSTATION 4000 kVA									350,000					
			ENGINEERING								50,000						
			ADD'L TRANSFORMER 2000 kVA														
			ENGINEERING														
10	CHEMICAL TREATMENT	CHEM. BLDG.	NEW STORAGE BUILDING	ULTIMATE								435,000					
			ENGINEERING								65,000						
11	CONTROL	COMPUTER	CENTRAL & REMOTE P.S. UNITS	ULTIMATE								670,000					
			ENGINEERING							80,000							
12	MISCELLANEOUS	ADMIN. BLDG. & ROADWAYS	ALTERATIONS TO EXISTING	ULTIMATE								650,000					
			ENGINEERING								100,000						
	TOTAL CONTRACT AMOUNTS									—	—	4,630,000					
	ENGINEERING									80,000	590,000	—					42
	TOTAL COST PER YEAR				—	—	—	—	—	80,000	590,000	4,630,000	—	—	—	—	42
	SUBSIDIES									12,000	89,000	695,000					6

No.	Revisions	By	Date

Drawn By:
R. MacLea

Checked By:
A. BOWERS

Date:
JULY 1982

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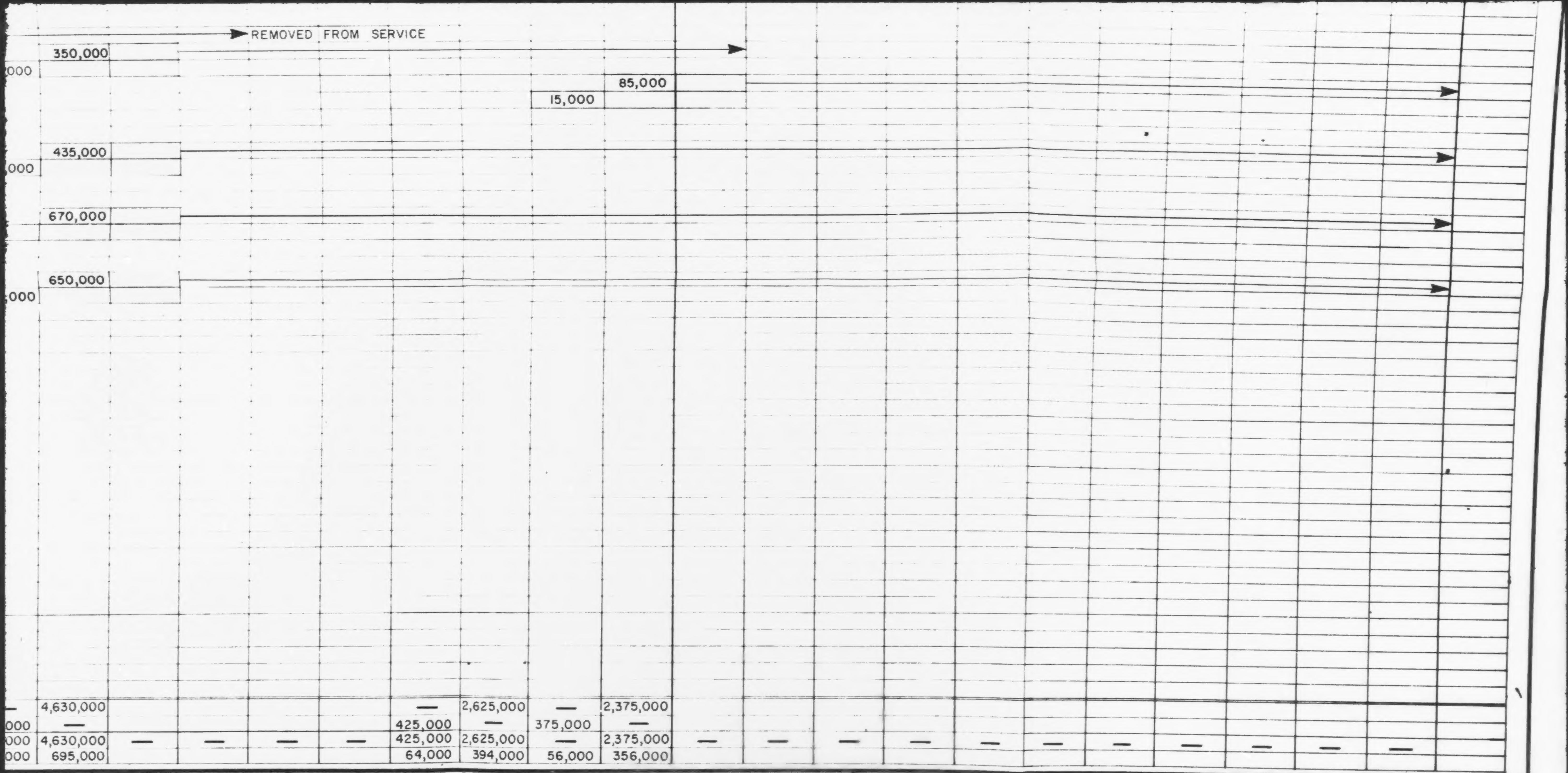

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The Regional Municipality of Halton

DEPARTMENT OF PUBLIC WORKS

OAKVILLE WATER
PURIFICATION PLANT

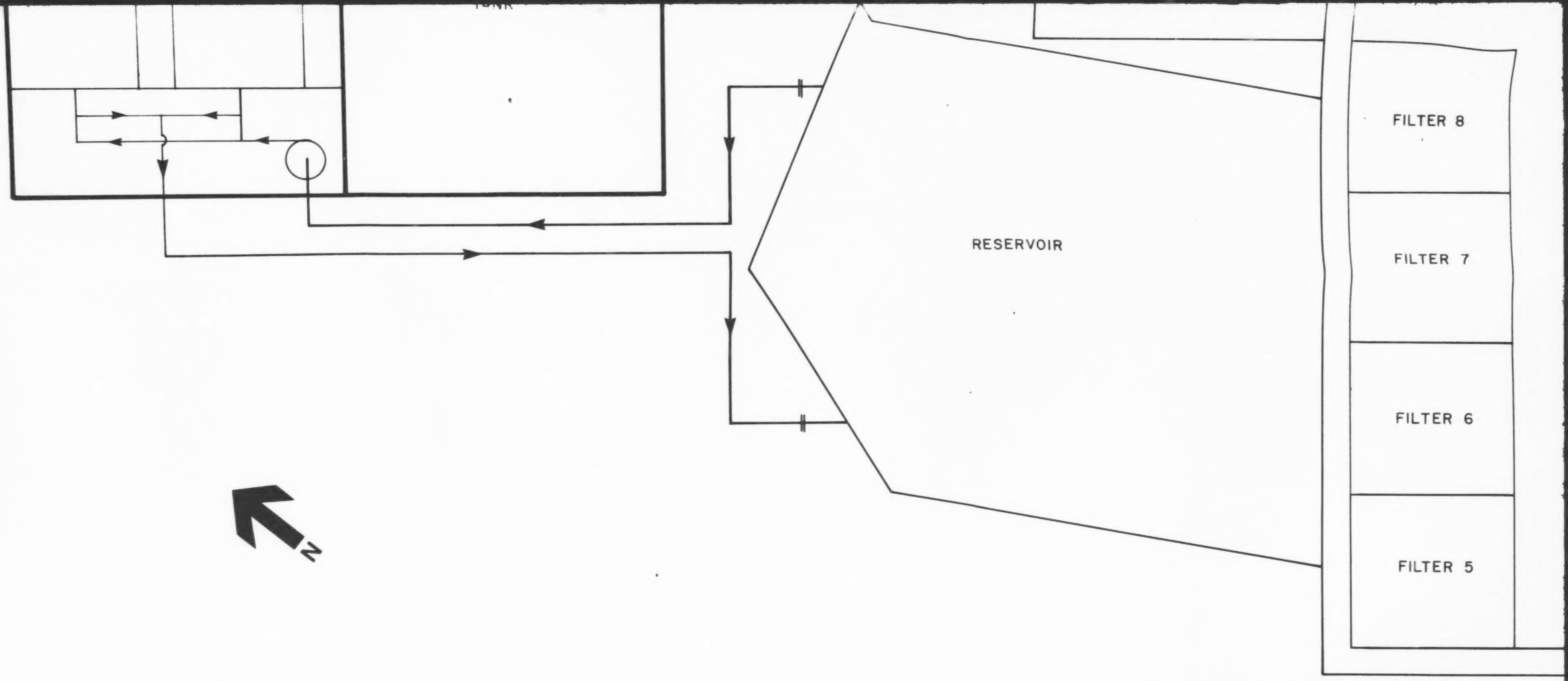
Drawing Title:

WATER PURIFICATION PLANT DEVELOPMENT PROGRAMME

All Dimensions Are In Millimetres
All Elevations Are Geodetic Metric

Scale:

Drawing No. R2


RESERVOIR
FILTER 8
FILTER 7
FILTER 6
FILTER 5
N

No.	Revisions	By	Date

Drawn By: R.M.

Checked By: A.J.B.

Date: JUNE 1982

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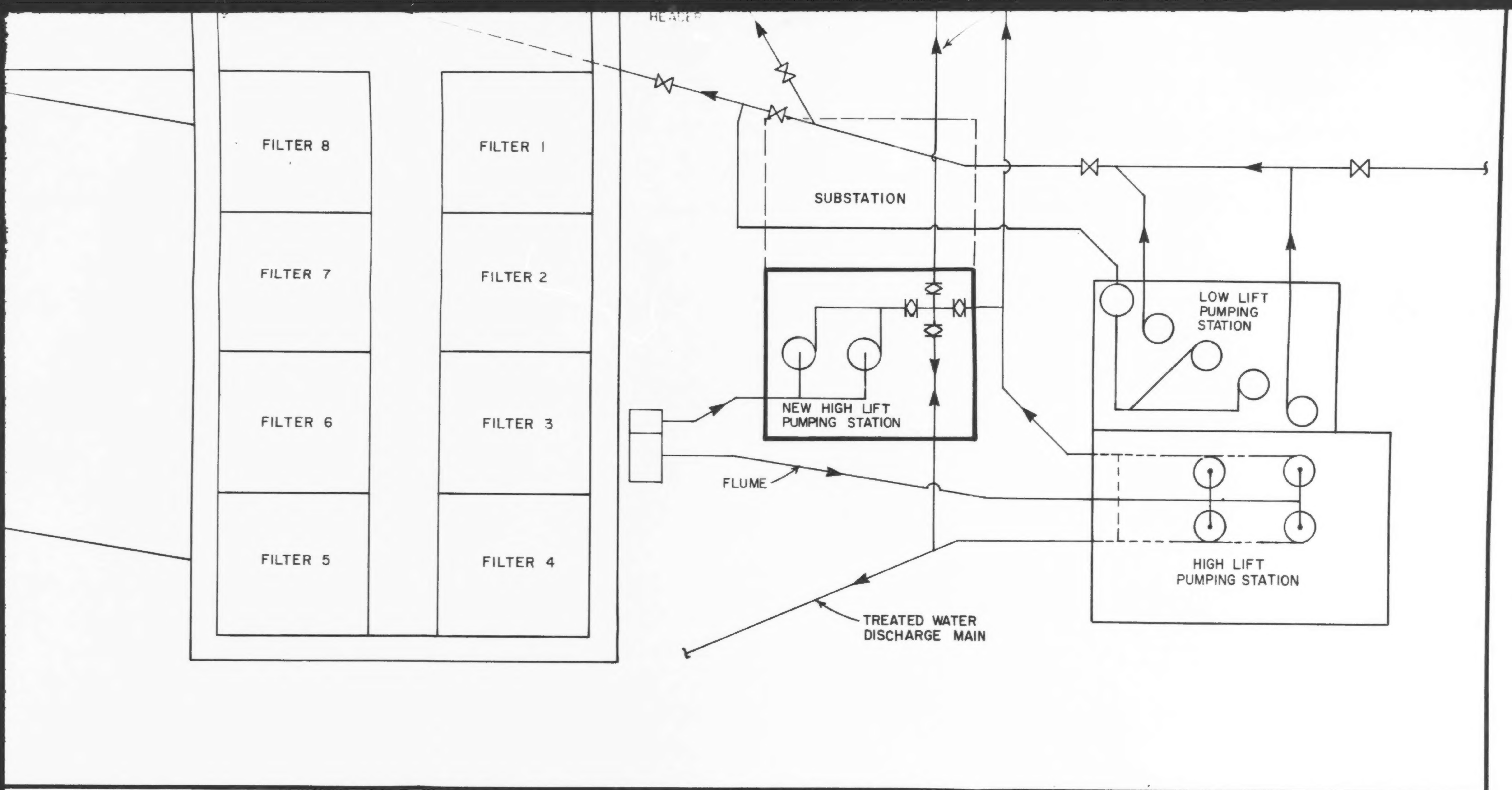
FILTER 8
FILTER 1
FILTER 7
FILTER 2
FILTER 6
FILTER 3
FILTER 5
FILTER 4
SUBSTATION
NEW HIGH LIFT
PUMPING STATION
LOW LIFT
PUMPING
STATION
FLUME
HIGH LIFT
PUMPING STATION
TREATED WATER
DISCHARGE MAIN

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DEPARTMENT OF PUBLIC WORKS

OAKVILLE WATER
PURIFICATION PLANT

Project Title: : OAKVILLE W.P.P. UPDATE REPORT

Drawing Title: SITE LAYOUT - FUNCTIONAL ARRANGEMENT

All Dimensions Are In Millimetres
All Elevations Are Geodetic Metric

Scale: APPROX. 1: 200

Drawing No. **R3**

POOR

QUALITY

ORIGINAL